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OF
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DECEMBER, 1925

Featuring

Modern Conceptions
Concerning Foundation
Engineering

Report on Metropolitan
Water Supply



Vol. XII

No. 10

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CONTENTS

PAPERS AND DISCUSSIONS

	PAGE
Modern Conceptions Concerning Foundation Engineering. By Dr. Charles Terzaghi	397

OF GENERAL INTEREST

Report on Metropolitan Water Supply	440
Boston's Earthquake Hazard	445
Motor Vehicles on Massachusetts' Highways	446
A. S. T. M. Standards	447
Proceedings of the Society	447

CONTENTS AND INDEX—Volume XII, 1925

Contents	iii
Index	vii

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MODERN CONCEPTIONS CONCERNING FOUNDATION ENGINEERING

BY DR. CHARLES TERZAGHI*

(Presented November 18, 1925)

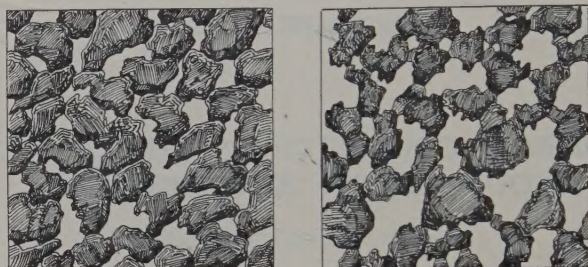
THE problems of foundations are as old as any with which the structural engineer has to deal. However, structural engineering became during the last one hundred years an elaborate and very intricate science, while in the vast field of earthwork engineering nothing has been produced except a set of pile-driving formulæ and an academic theory of earth pressure against retaining walls. Symbolically speaking, these theories fit into the frame of modern civil engineering as Trinity Church fits into Broadway. The structure is very venerable, but its foundations are just strong enough for supporting a one-story building. Hence, before erecting a more elaborate structure, one has to dig deeply below the old foundations and to provide a more substantial substructure.

This has been done by careful study of all those physical factors which cause the difference between the soils the engineer has to deal with and the ideal material invented by the authors of our classical earth-pressure theories. These factors can be assembled in three groups: first, the forces acting at the points of contact between the soil grains; second, the strain effect produced in soils by external forces; and third, the effect of time on the development of strain.

STRUCTURE OF SOILS. — All these physical factors are intimately connected with the structure of the soil. For this reason I prefer to start with a discussion of the structure. Up to this time most of the theoretical investigations have been confined to a study of cohesionless sands. The structure of sands ranges between two extreme limits, — the

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dense granular (Fig. 1 (a)), and the loose granular structure (Fig. 1 (b)). For sands with uniform grain size the two types of structures correspond to a volume of voids of about 26 per cent (densest) and 47 per cent (loosest structure). Structures corresponding to a volume of voids of more than about 47 per cent are unstable and apt to suffer



(a) Dense granular

(b) Loose granular

FIG. 1. — Structure of cohesionless sands

spontaneous changes associated with grain settlement. One of the most conspicuous properties of the sand resides in the fact that a sand with a loose structure cannot effectively be compacted except by processes involving violent jarring of the material. Static pressure uniformly applied all over the surface of the material has but little compacting effect. On the other hand, the most effective method of changing a dense structure into a loose one consists in sending through the material a strong upward current of water, — a process which takes place in practice if pumping water out of an open caisson.

Many engineers believe that the density of a sand loosened by pumping resumes its original value after pumping has been stopped. For investigating whether such assumption is justified the following test was made: a flow of water was sent in an upward direction through a sample of sand with a dense structure, while gradually increasing the hydraulic gradient. The quantity of water percolating through the sample increased in direct proportion to the hydraulic gradient, indicating that the structure of the sand remained practically unchanged (Fig. 2). However, at a certain hydraulic gradient ranging between 0.83 and 1.16, according to the density of the sample, the ratio

$$\frac{\text{percolation}}{\text{hydraulic gradient}}$$

suddenly went up; that means the structure of the sand became loose and remained so while the value of the hydraulic gradient further

increased. However, when gradually lowering the hydraulic gradient below the critical value mentioned, the ratio

$$\frac{\text{percolation}}{\text{hydraulic gradient}}$$

practically remained unchanged, which indicates that the structure of the sand remained loose (Fig. 2, dotted line). This conclusion was confirmed by the following observation: A sample of sand with a volume of voids of 50 per cent was charged with a load of 700 pounds per square inch at lateral confinement. In spite of the great intensity of the pres-

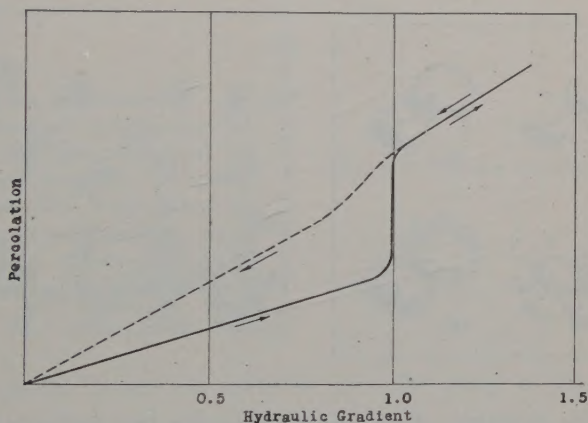


FIG. 2.—Effect of upward current of water on permeability of cohesionless sand

sure, causing 4.6 per cent of the grains to break, the volume of voids was merely reduced to 43.7 per cent, while by jarring and slight tamping the volume of voids of another sample of the same kind was reduced to 40 per cent. We say the structure of the sand is "conservative." That means the sand tends to retain the original arrangement of its grains even under considerable pressure.

STRUCTURE OF VERY FINE-GRAINED MATERIALS.—It was mentioned that the structure of a sand is not stable unless its volume of voids is smaller than about 47 per cent. If at the points of contact between the individual grains no other force should act but the friction proportional to the pressure, no looser sediment could exist in Nature. However, experience shows that there are many deposits of mud and of silt of which the volume of voids considerably exceeds the limiting

value of about 50 per cent. This fact induced us to investigate the interaction between soil grains more in detail.

According to the classical theory of earth pressure, the only force acting at the points of contact of the soil grains should be the frictional resistance, which in turn should be proportional to the pressure, regardless of whether the soil is dry or immersed in water. At a pressure zero this force ought to be equal to zero (Fig. 3 (a)). However, this conclusion is only approximately true, because even if the pressure exerted by one surface on to another one is equal to zero, and if in addition the grains are immersed as in Fig. 3 (d) (the surface tension of the water being completely excluded), the spheres of attraction of at least

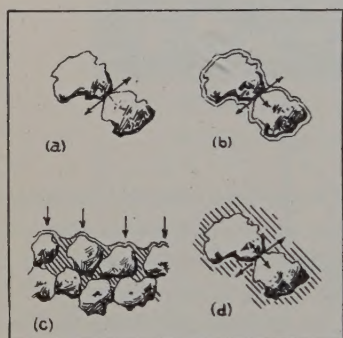


FIG. 3. — Interaction of soil particles
 (a) According to classical theory.
 (b) In contact with air, held together by adsorbed water film.
 (c) Voids filled with water, the water stressed by surface tension.
 (d) Soil completely immersed, no stress in the void's water.

two molecules overlap. As a consequence the two surfaces are chained together by at least one intermolecular bond (Fig. 3 (d)), which is, according to Chatley, for clay molecules, of the order one-millionth of a dyne, one dyne being equal to about one-thousandth of the force represented by the weight of one gram. This force seems to be insignificant. However, if we consider the fact that the weight of a coarse silt particle is not greater than the force represented by one intermolecular bond, the importance of the existence of this force becomes at once obvious.

Suppose a mass of grains be shaken up with water, thus forming a suspension, and, after that, be allowed to settle under the impulse of the force of gravity. As long as a sinking particle is completely surrounded with water its movement is a translatory one, because it travels under the influence of a single force — the force of gravity — through a homogeneous material, — the water. However, the very moment the particle touches the surface of the layer of the sediment covering the bottom of the vessel two new forces come into play. These forces

are the resistance of the ground against penetration (reaction) and the intermolecular bond acting at the point of contact between the sinking particle and the surface of the particle on whose surface it settles. If the particle has the size of a sand grain the intermolecular bond is negligibly small as compared with the weight of the grain. The weight and the reaction both acting on the grain form a couple (Fig. 4). Under the influence of this couple the particle starts to roll down the surface of the sediment until it comes to rest at the bottom of the depression located next to the point of first contact. The structure of the sediment thus formed will be of the type Fig. 1 (b) (loose granular structure). By thoroughly shaking the vessel which contains the sediment the

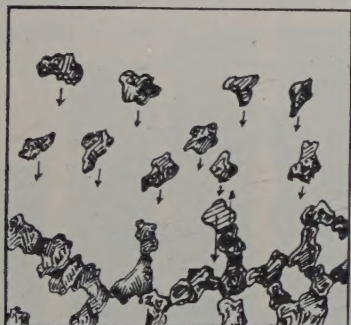


FIG. 4. — Process of sedimentation

structure passes from the state *b* into the state *a* (dense granular structure).

On the other hand, if the particle is very small the intermolecular bond is strong enough to overcome the tendency of the grain to overturn (effect of the couple weight $\times$ lever arm). Hence the particle remains in the position where it was when it first struck the bottom. The same is true of each succeeding particle. The individual particles are kept together by the intermolecular bond, which acts just like a true cohesive bond, and the structure of the sediment becomes of the type Fig. 5 (a) (honeycomb structure). The maximum volume of voids corresponding to this structure was found to be approximately equal to 80 per cent.

Let us finally assume the particles to be of colloidal size. Such particles remain in suspension forever unless they are precipitated by means of adding to the suspension a few drops of an electrolyte. In Nature the electrolyte is represented by the salts present in the water of the ocean or washed into the water of the rivers. Before the electrolyte was added the particles moved with considerable speed through the

liquid (Brownian movement) and repelled each other on account of their electric charge. The electrolyte neutralizes the electric charges without eliminating the physical cause of the Brownian movements, hence the particles collide in the liquid, and as soon as two particles collide the intermolecular bond comes into action and the particles stick together. Thus flocks are formed, with a structure of the type Fig. 5 (b). While the flocks sink down additional particles are picked up on their downward paths (flocculation or coagulation), and at the bottom of the vessel the flocks build up a sediment of the type Fig. 5 (a), with the only difference that there will be a flock in the place of each individual grain. The resulting structure has been called a "flocculent" structure (Fig. 5 (b)). As the maximum ratio between the volume of liquid and the volume of solid corresponding to a honeycomb structure of the type Fig. 5 (a) was found to be approximately equal to 4 (volume of voids

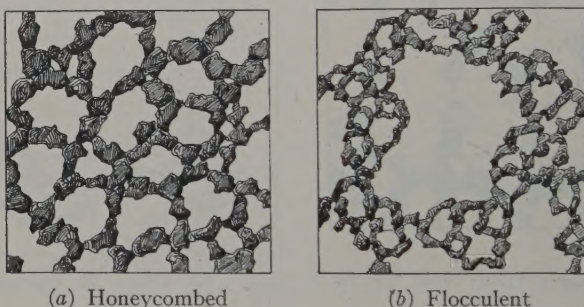


FIG. 5. — Structure of very fine-grained sediments

of 80 per cent), the maximum ratio between the volume of liquid and the volume of solids of a sediment of the type Fig. 5 (b) ought to be approximately equal to $4^2=16$, corresponding to a volume of voids of 94 per cent. This conclusion was confirmed by experiment.

Since the bond per point of contact is independent of grain size, while the number of bonds per unit of volume of the sediment increases with decreasing grain size, one ought to expect that the effect of the existence of the bond on the volume of voids should increase with decreasing grain size. In order to verify this conclusion I performed sedimentation tests with powders of different fineness extracted from mixed-grained aggregates by wet mechanical analysis. The results of some of these tests are presented in Fig. 6. Each one of the four cylinders shown in the figure contained precisely the same quantity of powder. In each cylinder the upper line indicates the surface of the sediment after it was allowed to settle for a couple of days, and the lower line is

the surface of the sediment after violently jarring the sample. For coagulated sediments (not shown in the drawing) the ratio between the volume of soil and the liquid was found to range between 16 and 20, in fair agreement with the theory.

If the powders are not immersed, but either wholly or partially in contact with air, the phenomenon is still more complex, because

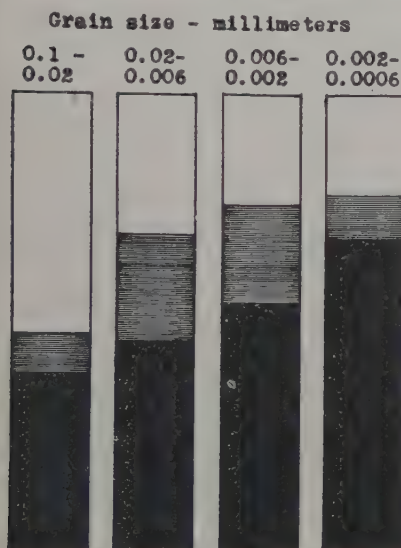


FIG. 6. — Space occupied by unit of weight of quartz powders with different grain size.

Upper lines: Surface of deposit after sedimentation in quiet water. *Lower lines:* Surface after shaking and jarring the sample.

the surface tension of the water becomes a factor of importance (Fig. 3 (b) and (c)). Since the effect of surface tension has been quite recently described in a paper in "Engineering News-Record,"* it is sufficient to mention that the pressure produced by the surface tension of the capillary water ranks among the most gigantic forces the engineer has to deal with.

STRAIN EFFECT. — A study of the strain effect produced by external forces in soils has led to the result that the relations between stress and strain in soils are almost as simple as they are for solid granular bodies. The only difference which was found to exist is this: for solids the modulus of elasticity and the resistance against shear is a constant, merely depending on the nature of the material. For powders both modulus of elasticity and resistance against shear increase in direct proportion with the intensity of the internal pressure.†

* "Engineering News-Record," November 5, 1925, p. 742.

† "Engineering News-Record," November 12, 1925, p. 796.

If c denotes a constant depending on the character of the soil, and p the internal pressure, the modulus of elasticity E of the soil was found to be equal to

$$E = cp.$$

This simple fact covers the vast field of stress-strain relations in soils, provided the voids of the soil are filled with air.

TIME EFFECT. — However, if the voids are filled with water we are obliged to consider a factor absent both in the mechanics of soils and of dry powders. Suppose the strain effect consists in compression. Compression means a decrease in volume. Both water and grains are practically incompressible, hence the compression involves a flow of water through the voids from the interior of the sediment towards the surface. No flow of water through porous bodies is possible without a hydraulic gradient. Hence the external pressure produces first a hydraulic excess pressure in the voids' water, and the speed with which the strain effect takes place is determined by the speed with which the water can escape towards the free surface. In coarse-grained powders the resistance against flow is very small, and as a consequence the strain effect produced by external forces occurs almost as rapidly as it does in solids. However, the smaller the grains the greater is the resistance the percolating water has to overcome, and for colloidal soils this resistance produces a lag of compression of months or of years, depending on the degree of permeability and the thickness of the deposit. A theoretical study of this phenomenon has furnished the following result: let —

k be the coefficient of permeability of the soil

a change in water content per unit of volume at a change in pressure equal to unity

z the vertical distance of a point (P) from the surface on which the external pressure acts

$\frac{\delta w}{\delta z}$ the hydraulic gradient at that point, and

$\frac{\delta w}{\delta t}$ the change of the hydrostatic pressure with the time at the same point.

The relation which exists between the time and the hydrostatic excess pressure w , acting in the capillary water at the point P located in a depth z below the surface, was found to be determined by the equation —

$$\frac{k}{a} \times \frac{\delta^2 w}{\delta z^2} = \frac{\delta w}{\delta t}$$

difference between the water pressure at the surface and the water pressure in the interior, which pressure difference in turn causes the water to flow from the interior to the surface. Due to its proximity to the free surface the upper part of the layer of clay consolidates first, and the consolidation gradually proceeds from the surface towards the bottom. Simultaneously the hydraulic gradient gradually decreases and finally becomes equal to zero. This process lasts the longer, the less permeable the clay is. The abscissæ of the curve ab represent the pressure which acts at a time t_1 after the pressure p_1 was applied, and the difference $w = p_1 - p$ gives the hydrostatic excess pressure acting simultaneously in the voids' water of the clay at the same point where p was measured.

The thermodynamic analogy of this process is as follows: A layer of homogeneous material completely isolated against heat radiation except on its upper surface is supposed to have a uniform temperature. The increase in pressure corresponds to a sudden drop in temperature of the space located above the layer. Due to the difference in temperature the heat flows from the interior of the layer towards the surface. Since the top layers obviously cool out first, the cooling process proceeds from the surface towards the isolated bottom. Suppose the numerical values of k and a for the clay are equal to the numerical values of the coefficient of heat conductivity and of the specific heat for the hot body, and that in addition we choose the scale for the abscissæ in Fig. 7 such that the increase in pressure represents the drop in temperature of the space located above the hot plate also. Under these conditions and basing on the thermodynamic analogy, we can make the following statement: the curve which represents the pressure acting in various depths of the layer of clay, for any definite time t_1 (Fig. 7, curve ab) after the pressure p_1 was applied, represents the curve of temperature distribution in the cooling plate for the time t_1 also. That means the abscissæ of the curve ab represent not only the difference between the pressure acting in the clay at a time t_1 and the original pressure p_0 , but they also determine the difference between the original temperature of the hot plate and the actual temperature for the same time t_1 .

The thermodynamic analogy gives full information on all time-settlement phenomena apt to occur in connection with foundations on clay. Up to this time theoretical investigations dealt almost exclusively with cohesionless materials, and the clays were considered as almost unfit for theoretical treatment. With the new information at our disposal it was found that it is far easier to theoretically analyze the behavior of homogeneous clay under load than to investigate the analogous behavior of cohesionless sands.

CONSOLIDATION OF MUD DEPOSITS. — One of the most interesting applications of the thermodynamic analogy to soil phenomena concerns the gradual consolidation of silt and mud deposits under the influence of no other than their own weight. Suppose a mud deposit has been formed by sedimentation at the mouth of a river or artificially by dumping material excavated by hydraulic dredges. The deposit settles under the influence of its own weight. How long will it last until equilibrium is reached, that means, until the settlement of the deposits will cease? From the differential equation of the time effect we learn that the speed of the process depends on the coefficient of permeability k and on the

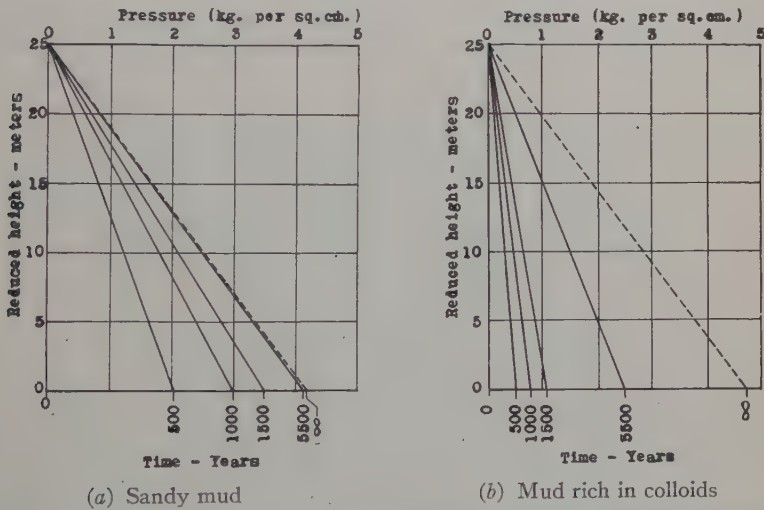


FIG. 8. — Consolidation of mud-deposit under the influence of its own weight.
The deposit is supposed to be located under water

compressibility α of the material. The thermodynamic analogy of the drainage process consists in the cooling out of a plate whose original temperature increased from the surface towards its bottom. The difference in temperature between the surface and the bottom gradually decreases and finally becomes equal to zero. The settlement corresponds to the shrinkage caused by cooling. Fig. 8 (a) and (b) show the successive stages of the consolidation process for two different materials, — a sandy mud and a very colloidal mud. The abscissæ of the dotted lines give the total pressure due to overburden for different depths below the surface. The full drawn lines divide the abscissæ of the dotted ones into two parts of which the right one indicates the hydrostatic excess

pressure acting in the void's water at a definite time, and the left one the pressure acting in the solid part of the deposit at the same time. The figure shows how slowly the hydrostatic excess pressure acting in the void's water decreases. Since the internal friction — that means, the shearing strength of the mud — is equal to the product of the co-efficient of internal friction and the pressure acting in the solid part of the mud-water mixture, the deposit behaves in an early stage of consolidation almost like a very viscous liquid.

Fig. 9 shows the pressure distribution in a delta deposit advancing towards the ocean at a rate of 3 feet a year, the top layer drying out by evaporation. In the lower part of the deposit located below the coast line, the capillary water stands under a positive hydrostatic excess pressure. Due to this excess pressure the friction acting within the

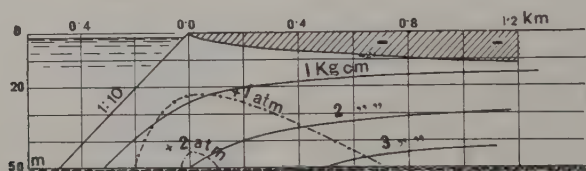


FIG. 9. — Hydrostatic pressures acting in the void's water of a clayey delta-deposit. The top layer became partially consolidated by evaporation, while the lower parts of the deposit are still soft

deposit is small, and the material is apt to break out towards the water front. Such submarine landslides are familiar to the geologist, and as far as their physical causes are concerned they are identical with the failure of hydraulic-fill dams with cores rich in clay. On the other hand, the top layer of the deposit is thoroughly compacted by the capillary pressure produced by evaporation and the void's water is in the state of tension (negative hydrostatic pressure). According to the laws of hydrodynamics the water drains from the region of positive hydrostatic pressure towards the zone of negative pressure, but, due to the low degree of permeability of the material, the drainage process may last several thousand years. In a similar way, the peripheral part of the core of a hydraulic fill dam, consisting of clayey material, may partially consolidate by evaporation, while the core remains soft. If one digs through the top layer of a deposit of the type Fig. 9, a canal, the bottom of the canal would furnish some water, while the slopes, located within the top layer, would absorb water and swell.

The graph Fig. 9 was obtained by theory, based on the physical constants which have been determined for muds from the water front of Constantinople. At the time the graph was plotted no empirical data concerning the actual pressure distribution in mud deposits were available. However, half a year later, when engaged in preliminary work for the foundation of a factory on one of the mud deposits of the Golden Horn, I had an opportunity for checking the theory against actual conditions. The results of this investigation are shown in Fig. 10. The lower part of the deposit represented on the drawing is of pre-historic age. The mud above it was known to be about one hundred years old. The abscissæ of the dotted lines represent the consistency the mud would have in the state of hydrostatic equilibrium. The data required for plotting these lines were obtained by testing the mud samples in the laboratory. The surface of the mud deposit is covered

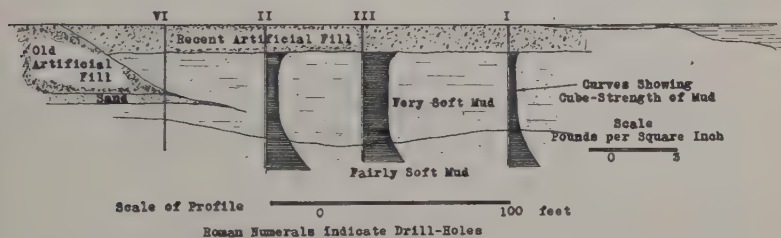


FIG. 10. — Cross-section through a mud-deposit at the coast of the Golden Horn, Constantinople. The width of the shaded areas indicates the consistency of the muds at different depths below surface

with a thin layer of recent, artificial fill. The shaded areas represent the actual consistency of the mud as measured by the cube strength of samples. From comparing the two sets of lines one learns that more than three-quarters of the dead weight of the mud is still compensated by hydrostatic excess pressures. As a consequence the settlement of the ground will continue for several hundred years, under the influence of no other than its own weight. According to the thermodynamic analogy this phenomenon corresponds to the fact that the shrinkage due to cooling of a hot plate continues as long as the temperature of any one of its parts is higher than the temperature of the space above it. Since settlements of this kind cannot possibly be stopped, property had to be abandoned and another site for the factory was selected.

BEARING CAPACITY AND LOADING TESTS. — Before I was invited to take charge of this piece of work the operations had been conducted

by another engineer. This gentleman made loading tests on the ground and several loading tests on piles. As the results of the tests were considered to be fairly satisfactory, \$15,000 worth of piles were driven into the ground. I myself was not consulted by the owners until a self made but experienced architect called their attention to the fact that the enterprise seemed to be somewhat risky. This incident leads us to inquire which conclusions can be drawn from the results of loading tests. Ingenious devices have been proposed and employed for making such tests, but the textbooks are remarkably reticent concerning the use we can make of the results.

Theory made it possible to predict with a fair degree of accuracy the distribution of the pressures in the ground below a foundation.

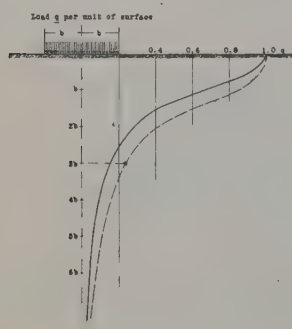


FIG. 11. — Distribution of vertical soil-pressures below the center of a circular area, charged with a load q per unit of area

In Fig. 11 the full drawn line gives the theoretical pressure distribution for the underground of a circular foundation with a diameter $2b$, and the dotted line, the pressure distribution as determined by independent experiments carried on by the Pennsylvania State College. However, far more important are the results furnished by theory concerning the settlements. Suppose a loading test has been executed on an area of 2.2 square feet, and the settlements produced by a load of 1 ton per square foot were found to amount to one-quarter of an inch. How big would be the settlement of a building supported by a raft foundation 30 feet wide, exerting on the ground the same pressure per square foot? The information obtained by theory for the interpretation of the results of loading tests performed on homogeneous ground (clay with a uniform consistency or sand with a fairly uniform density) are represented by the graph Fig. 12. According to this graph the settlement of the raft foundation may be anything between the limiting values of three-eighths of an inch and about 5 inches, depending on the character of the soil material.

The shape of the upper curve (very cohesive material) has been checked by the results of independent experiments made by *Goldbeck* and others. The lower curve corresponds to the settlement of loads supported by perfectly cohesionless material, and its trend is known approximately only. At Massachusetts Institute of Technology we plan some experiments for investigating the character of this lower limiting curve more in detail. The graph explains the appalling contradictions which exist between the experts' opinions concerning the conclusions one is allowed to draw from the results of loading tests. These opinions obviously depend on the character of the underground of the localities where the experts have made their observations, and no general understanding

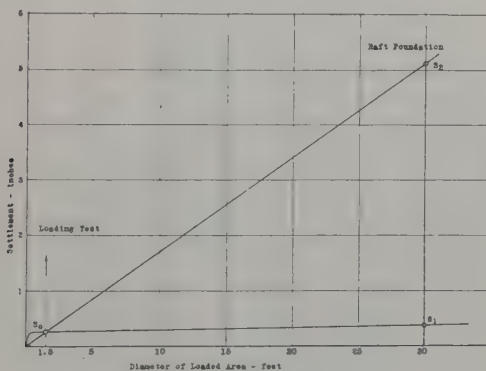


FIG. 12.—Relation between diameter of loaded area and the settlement produced by a load of 1 ton per square foot. Upper curve for homogeneous, plastic clay; lower curve for cohesionless sand

can possibly be reached until the theoretical principles on which the graph Fig. 12 is based become self-evident to everybody engaged in foundation work.

On account of the important influence of unequal settlement on the stresses in statically indeterminate structures, the forecast of the settlements is one of the most important problems we have to deal with. No such forecast can possibly be made unless we are in a position to plot, for any individual case we have to deal with, a graph which approximately represents the relation between the diameter of the loaded area and the amount of settlement. I do not think we will ever be able to plot such a curve unless we explore the character of the ground not only by loading test but by driving a test pile also. Such tests cost money,

and the question arises whether the expenditure involved would be justified or not.

In cities with a fairly solid underground consisting of dense sands or gravels where settlements amount to not more than half an inch, the money invested in elaborate tests would duly be considered an unwarranted expenditure. Empirical rules derived from previous experience are fully sufficient. However, there are localities where settlements from a couple of inches up to one foot are inevitable, and in such localities the situation is altogether different. As examples I want to mention Constantinople in Turkey, and Trieste in Italy. Due to the inability of the engineers to estimate the settlements in advance, the building laws of such cities are almost a farce. Officially every building has a factor of safety of 4, but the secondary stresses due to unequal settlement are passed over in silence. If the beams and girders of these buildings could speak and tell about the stresses they have to sustain we would experience startling revelations. We would realize that the factor of safety of the superstructures ranges anywhere between unity and 4 according to the type of building and the locality. In Constantinople I had ample opportunity to get familiar with safety conditions as affected by excessive compressibility of the underground. It is almost incredible that such conditions can exist in the enlightened twentieth century, not only in Constantinople but in many other cities. Our uncertainty concerning the amount of unequal settlement upsets for certain districts all our principles concerning the safety of structures, and construction work has under such conditions far more the character of a gamble than the overground construction work has had in the Middle Ages before the principles of applied mechanics were established. In the whole field of civil engineering there are hardly any efforts as urgently needed as the efforts for predicting the amounts of settlement, and positive achievements in this line would be apt to lead to a fundamental modification of our inadequate building laws.

PILE FOUNDATIONS. — However, for the time being, whenever the quality of the ground seems to be deficient, we simply prescribe the universal remedy of the foundation expert, — the driving of piles. This remark leads us into one of the queerest quarters of the queer field of foundation engineering. For more than a century desperate efforts have been displayed to derive a pile-driving formula — that means a formula for computing the bearing capacity of piles from the penetration produced by the blow of the hammer. Judging from the intensity of these mental efforts one could believe that such a formula represents the Philosopher's Stone. In Constantinople, where pile driving is quite

a popular sport, I had an opportunity to make some interesting observations. The results of one of these observations are shown in Fig. 13 (a). This figure represents a friction pile driven into soft plastic mud with a fairly uniform consistency. By loading and pulling tests it has been found that the frictional resistance per unit of length of such a pile is almost constant for the whole length of the pile, and that the bearing capacity of the pile is but very little greater than the resistance

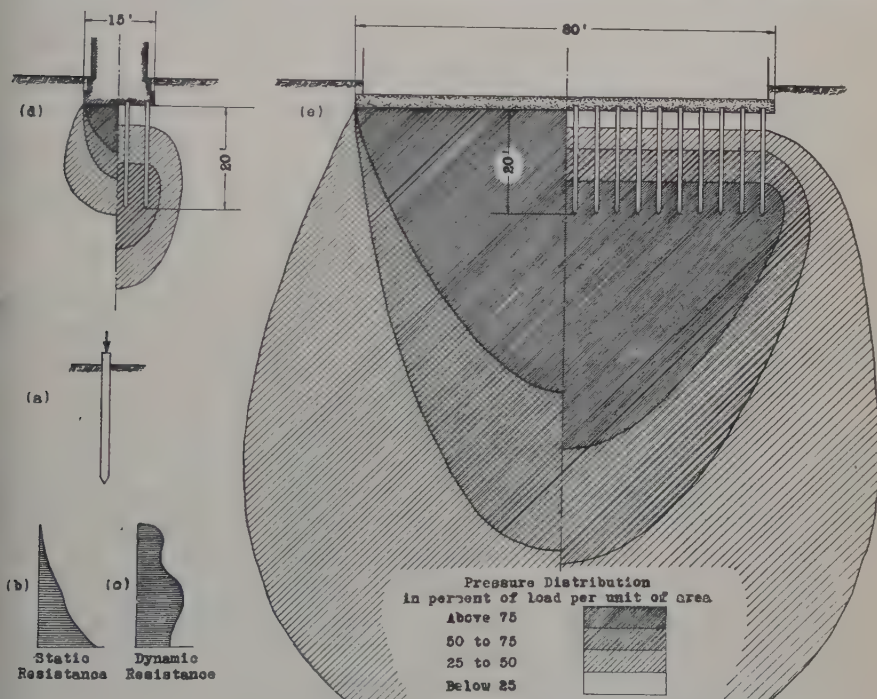


FIG. 13. — Pile driving resistance ((a), (b) and (c)) and the effect of the ratio between width of foundation and length of piles on the pressure distribution ((d) and (e))

of the pile against pulling. Hence the curve which represents the relation between the bearing capacity and the depth of penetration should be approximately a triangle, as shown in Fig. 13 (b). However, when computing the bearing capacity for various depths of penetration from the rate of penetration by means of any one of the pile-driving formulæ, I invariably obtained curves of the type Fig. 13 (c). Several hundred piles were driven into the mud deposit, and the penetrations produced by the blows were observed, but the discrepancies between the two

curves of bearing capacity was always as conspicuous as the one shown in the figure. Analysis of this fact and of other experiences of a similar kind has led to the following conclusion: We have to distinguish between dynamic pile-driving resistance — that means the resistance of the soil against rapid penetration of the pile — and the bearing capacity — that means the resistance against the penetration of the pile under static load. For piles driven into colloidal soils the two resistances have, as far as their physical causes are concerned, nothing in common, because the dynamic pile-driving resistance was found to act at the lower end of the pile and to be caused by the resistance against rapidly displacing the mud and the water, while the bearing capacity is almost exclusively due to the friction acting between the lower end of the pile and the surface of the ground, the resistance acting at the lower end being practically negligible. On the other hand, for certain granular soils the two resistances were found to be almost identical, both physically and numerically. Therefore no pile-driving formula of universal validity can possibly be obtained. This result is not surprising. Far more surprising are the strenuous efforts made to solve the insoluble problem, since an expenditure of \$200 for a loading test is sufficient to get an accurate individual solution of a problem whenever needed.

Unfortunately the problem of determining the bearing capacity of individual piles is of secondary importance as compared with the more complex problem of predicting the behavior of pile foundations on friction piles as a whole. The minds of the experts were so thoroughly occupied with the study of the behavior of the individual piles that the really essential part of the problem has received but little attention.

At equal factor of safety for the individual piles the behavior of a pile foundation on friction piles depends on two factors, — the nature of the ground and the ratio between the length of the piles and the width of the foundation. The nature of the ground determines in addition the functions the piles have to perform. Regarding the problem in question, the most important distinguishing features between the different kinds of pile-driving grounds were found to reside in the degree of compressibility, of permeability and of consistency of these grounds. According to this, and for briefly describing the characteristics of pile foundations, we may divide the soils into four different groups:

First: Permeable and Very Compressible Soils. — For such soils the chief purpose of the piles consists in compressing the ground, that means in reducing its volume of voids. The bearing capacity of the individual piles is a factor of minor importance. The piles serve their purpose the better the more violently the pile-driving process acts on

the soil and the more soil the piles displace. In loose cohesionless soils conical piles driven with a steam hammer seem to have the best effect. In loamy fills unfit for being compacted by vibrating action, the piles of the Compressol system can be used to advantage.

Second: Permeable, Unequally Compressible Ground. — Foundations of this type represent the ideal field for using conical piles of modest length, the function of the piles consisting in equalizing the unequal resistance of the upper strata. In Austria an engineer has devised a method for obtaining for each pile a penetration graph similar to the records furnished by seismographs. These graphs give full information concerning the variations in the resistance of the underground, and the distance between the piles is calculated accordingly. By using this method in connection with pile foundations on short, very conical piles, perfectly uniform settlement has been obtained for buildings constructed on soils with very unequal resistance.

Third: Clay and Mud Deposits in a State of Hydrostatic Equilibrium. — The hydrostatic equilibrium involves increase of the consistency with the depth, and the main function of the piles consists in transferring the pressure from the softer upper strata to more resistant lower ones.

Fourth: Clay and Mud Deposits of Uniform Consistency. — This class of soils includes a great number of deposits located at the water front of the ocean and of lakes. Since foundations on deposits of this kind are very common in harbors and harbor districts, some remarks concerning the effect of piles on the bearing capacity of such deposits may be justified. The information furnished by theory about the distribution of stresses underneath foundations supported by clay soils of uniform consistency are represented in Fig. 13 (d) and (e). In each one of these figures the left-hand section shows the stress distribution underneath a foundation without piles, and the right-hand section the effect of 20 feet piles on the stress distribution. If the length of the piles is at least equal to the width of the base, the effect of the piles is obviously very beneficial (Fig. 13 (d)). The piles reduce the intensity of the maximum pressure acting on the ground, and in addition they shift the zone of maximum stress from the surface down to the level where the lower ends of the piles are located. Up to this time very little is known concerning the effect on the bearing capacity of the depth below the surface at which the pressure acts, but it is believed this effect may be quite an important one. Some experimental investigations concerning this effect are planned, and will be started shortly. On the other hand, if the width of the foundation is considerably greater than the length of the piles (Fig. 13 (e)), the effect of the piles is practically nil

and the money invested in the piles represents an unwarranted expenditure. Some years ago a company intended to construct a powerhouse with a raft foundation supported by 500 reinforced concrete piles 25 feet long. I advised the company to sell the piles and to construct a simple raft foundation. However, since some of the piles were driven already, it was decided to drive all of them. Fig. 14 (a) shows the settlements of the corners of the new building supported by piles, and Fig. 14 (b) the simultaneous settlements of the corners of an adjoining building

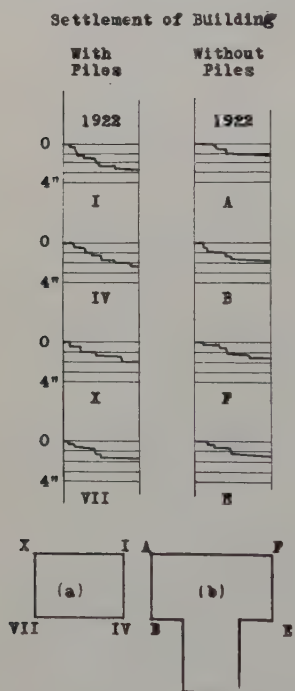


FIG. 14. — Settlement of the corners of two adjoining buildings, the building (a) supported by a pile foundation and (b) by a simple raft foundation without piles

supported by simple raft foundation and exerting on the ground the same pressure per unit of area as the building Fig. 14 (a).

Some time later I had to establish a structure with a narrow base on the same ground, and I proposed pile foundation. The piles were loaded with about one-third of their ultimate bearing capacity, and the settlement of the structure was so small that it could hardly be measured. The cause of the difference in behavior of the two types of structures may be learned by comparing Fig. 13 (d) and (e).

Foundations of Weirs on Permeable Ground. — This problem is one of the most difficult the foundation engineer has to deal with, because

it represents a problem of bearing capacity complicated by the hydrodynamic pressure exerted by the seepage water. The only way of solving a problem of this kind consists in starting from the simplest assumptions and in analyzing step by step all the factors involved. Fig. 15 (a) shows the cross-section through a single row of sheet piles, and Fig. 15 (c) the cross-section through a simple flat base dam. The water stored

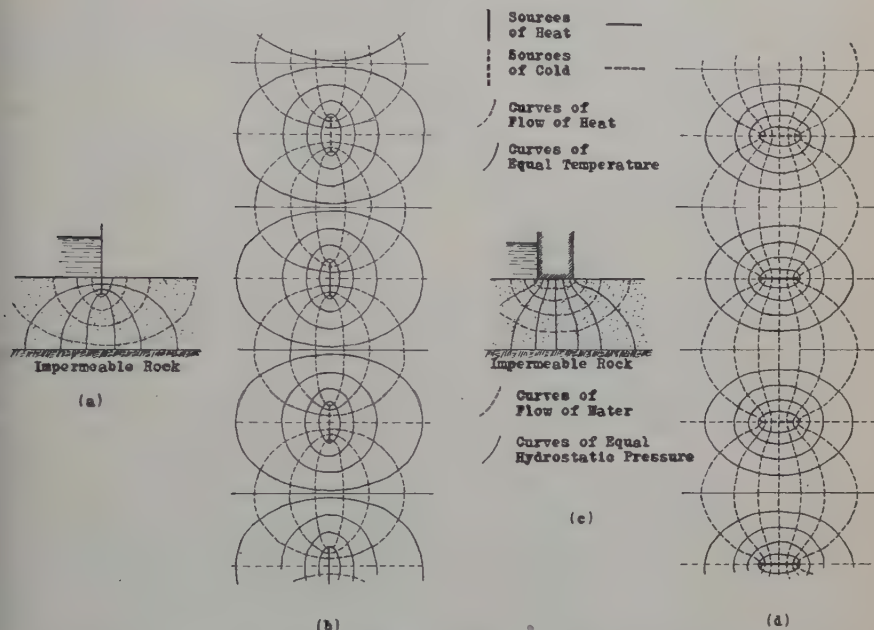


FIG. 15. — Lines of flow of seepage water through the permeable underground of weirs, compared with the lines of flow of heat from sources of heat toward localities of heat absorption

up behind these structures percolates through the ground and escapes at the downstream toe. The differential equation of any such flow is

$$\frac{\partial^2 \phi}{\partial x^2} = \frac{\partial^2 \phi}{\partial y^2}$$

where ϕ denotes the hydrostatic pressure. If we replace ϕ by temperature, we obtain the differential equation for the stationary flow of heat through homogeneous bodies. We say the lines of flow and the lines of equal hydrostatic pressure represent an isothermal system. The isothermal system representing the flow of the seepage water underneath a row

of sheet piles is geometrically identical with one section of the isothermal system produced by a series of sources of heat alternating with localities of heat absorption (Fig. 15 (b)). Fig. 15 (d) represents a similar thermodynamic analogy for the percolation underneath the base of a flat base weir. The geometry of the isothermal systems was established as early as 1859 by *Riemann*. Some forty years ago *Philipp Forchheimer* introduced the theory of isothermal systems into the field of theoretical hydraulics. In 1918 the same scientist succeeded in deriving the equations of the lines of flow shown in Fig. 15 (a) and (c), by using the thermodynamic analogy mentioned, and in addition he derived formulæ for calculating the seepage losses. Thus the geometric and the hydraulic part of the problem was solved. It remained to analyze the stresses produced by the percolating water and to determine their effect on the equilibrium of the underground. This problem formed part of my own investigations.

When flowing under head through permeable ground the water exerts on the soil a pressure. In every point of the underground the pressure acts in the direction of the flow, and its intensity is directly proportional to the local hydraulic gradient. Since both the direction of the flow and the hydraulic gradient are determined by the isothermal system, which is known, it was possible to compute the pressure effect produced by the water. Fig. 16 graphically shows this effect for a flat base dam and for different depths of impounded water. The intensity of the vertical component of the forces acting per unit of volume of soil is expressed by the degree of darkness of the colored areas, and the intensity of the horizontal component of the same forces by the difference in spacing of the horizontal shading lines. The graph discloses the following facts: within the area left blank the hydrodynamic upward force is greater than the weight of the soil, hence the soil included within these areas virtually turns into quicksand offering no lateral resistance. Against this unstable body of soil presses another body of soil acted upon by horizontal forces. As a consequence the equilibrium of the soil breaks down, the soil located underneath the base of the weir rushing downstream in a horizontal direction. If increasing the width of the base of the weir, the size of the quicksand body decreases but the quicksand condition continues to exist. By providing the weir with an impermeable apron a similar effect is produced. If loading the danger zone with broken stone, the soil is washed through the voids of the fill and the danger continues to exist. The only efficient remedy is this: to keep the soil in the danger zone down, by means of a graded filter. The

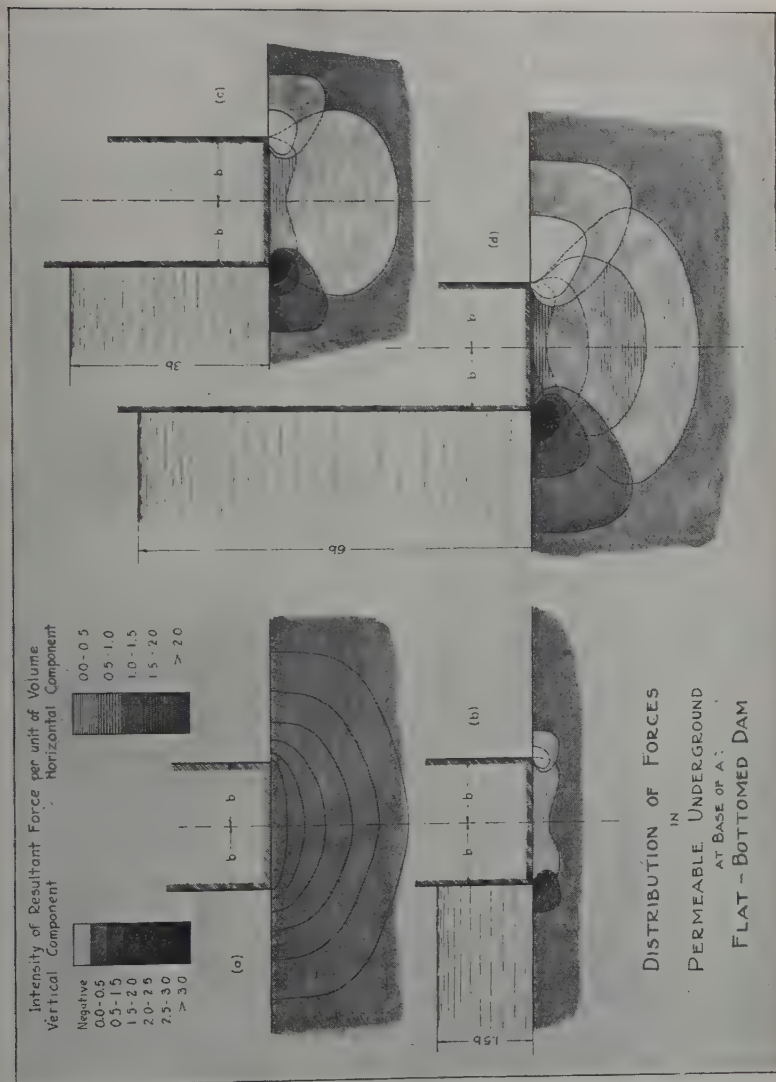


FIG. 16. — Graph representing the pressures produced by the flow of seepage water through the permeable underground of a flat-bottomed dam

filter safeguards the free escape of the seepage water, and at the same time it prevents the soil particles from drifting away.

In order to check both the theory and the conclusions, a series of tests was made. The results are presented in Fig. 17. The shaded triangles represent the cross-section of bodies of water equal in weight to the difference between the hydrodynamic upward pressure and the weight of the soil at the moment when the blowout occurred (computed by theory). The rectangles superimposed over the shaded triangles show the cross-section of bodies of water equal in weight to the weight of the filters applied above the danger zone. The figures show that the filter weight required for preventing the blowout to occur below a certain critical head of impounded water is somewhat smaller than the upward pressure, theoretically computed for the same critical head. If comparing the diagrams (c) and (d) (Fig. 16) one learns that extending the filter beyond the limit of the danger zone has practically no effect on the critical head. This result in turn indicates that the value furnished by theory for the width of the danger zone is fairly accurate.

The theory of the piping effect furnishes one of many instructive examples for what theory can achieve in the field of earthwork engineering. The traditional methods of designing foundations of weirs on permeable ground are as crude and uneconomical as are attempts to increase the bearing capacity of steel bridges without knowing in which members the maximum stresses are apt to occur. Using theory as a basis, we can confine our reinforcements to those parts of a structure where the danger exists.

Some years ago a weir of the Ambursen type near Pittsfield, Mass., failed due to piping. Basing my computation on the scarce data published concerning this accident, I estimated that the critical head must have been somewhere between 31 and 46 feet. The failure occurred at a head of 34 feet. At an expenditure of about \$20 per linear foot the factor of safety for the structure could have been increased from 1 up to 3.

Fig. 18 shows a photograph of the first weir designed by means of the theory of the piping effect. It has been built across the river Mur in the Alps. Due to the torrential character of this river most of the foundations located within its bed have been established on bed rock by means of compressed air. By using ordinary sheet pile foundation combined with a filter device, the costs of foundation of the new weir have been comparatively very low. Construction work was completed this spring. A few weeks later a catastrophal high water came down the river. A weir located several miles upstream failed, due to piping

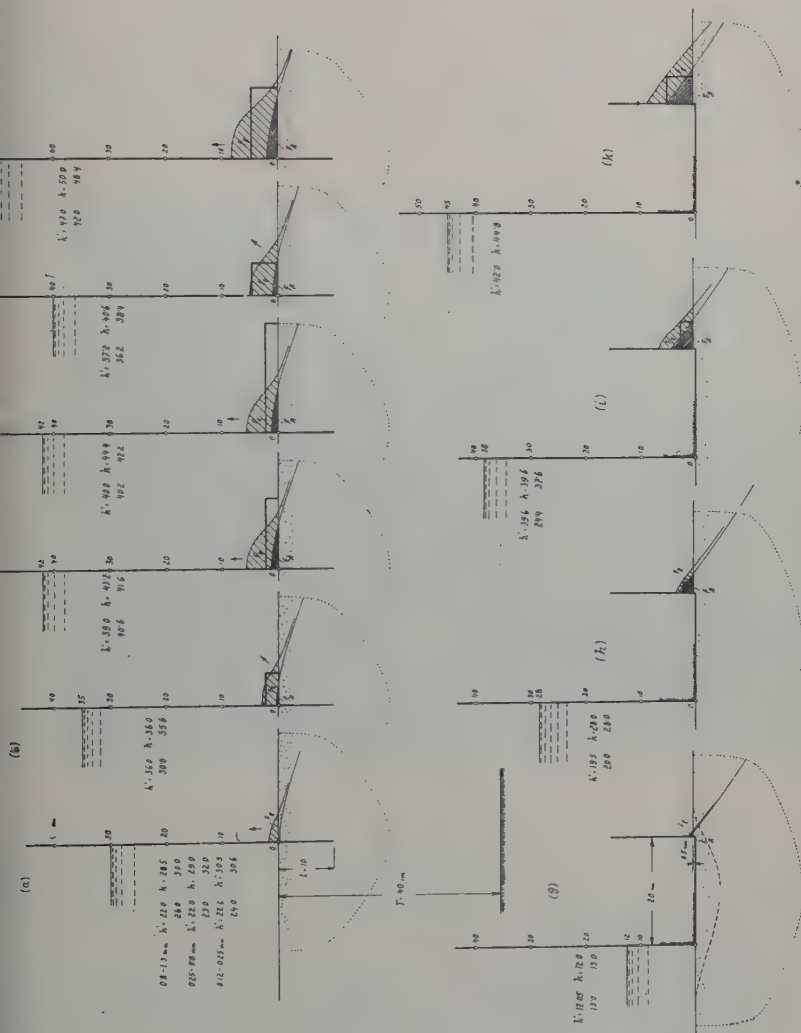


FIG. 17. — Effect of filter loads on the retaining capacity of a simple row of sheet piles ((a) to (f)) and of a flat-base dam ((g) to (k)), located on permeable underground. In each case the "blowout" occurred as soon as the surface of the impounded water reached the level indicated in the figure. In diagram (f) the dotted line indicates the bottom of the erosion canal produced by the "blowout"

effect. The new weir subsisted in spite of the gloomy prophesies issued from the quarter of the compressed air contractors, who for obvious reasons do not like the theory of the piping effect.

Gentlemen, at the outset of my lecture I compared the classical earth-pressure theories with Trinity Church. The urgent need for a more substantial building is unquestionable. What I have done has been to undercut the old foundations and to brace them against more resistant substrata. In so doing I was alone, with limited funds and without assistance. This fact may account for the rudimentary charac-



FIG. 18. — Weir with filter foundation across the River Mur, Austria

ter of what I could present. The results are promising, but the major part of the work remains to be done. Hence I wish to conclude my lecture with a few remarks concerning what should be done next for developing the young science.

For advancing a special branch of physics or chemistry, it is sufficient to provide funds, fellowships and to establish new laboratories. There are ample men who can do the work, because training in physics has been standardized. However, in soil mechanics the situation is far more complex. For producing valuable results in this field of applied science a man must be at home in at least five or six different sciences, some of them far remote from the field of civil engineering, and in addition he

must have a broad, practical experience or else he cannot discriminate between what is essential and what non-essential. The men who nowadays fulfill these requirements are exceedingly scarce, because their expertness has not been produced by systematic training but by patient self-education, guided by instinct rather than by a defined program. Without such men nothing worth while can be accomplished. We learned this fact from the cumulative results of related efforts made during the last ten years. Ten years ago we were more optimistic concerning this point. Furthermore, in physics or chemistry objectionable results if published do not harm, because they are pretty soon disproved. In earthwork engineering such publications are utterly detrimental, because very few engineers are equipped with the knowledge required for competent criticism.

Hence, what we need more than anything else are measures of an educational character. Young men who desire to engage in underground engineering should get the proper guidance and an opportunity to acquire all the knowledge required for competent work in their future field. Competent work in this field requires among others far more thorough training in applied geology, physics and physical chemistry than the structural engineer needs. The students who get this training would represent a generation of engineers better fit for shedding light over the obscure field of earthwork engineering than is ours. An *individual* may furnish the ideas, but the bulk of the work must be done by a properly educated *generation*. For this reason the attempt of the Massachusetts Institute of Technology to introduce soil mechanics into its program may be of greater importance for the future development of the most backward branch of civil engineering than any individual achievement.

Discussion

PROF. CHARLES M. SPOFFORD:* I do not wish to discuss the details of Dr. Terzaghi's paper tonight, but am pleased to take this opportunity to make a few remarks concerning Dr. Terzaghi's coming to the Institute of Technology. Last year I heard of the research work in soil mechanics that was being conducted by Dr. Terzaghi in Constantinople, and also ascertained that he was to have a year's leave of absence from Robert College. Appreciating perhaps as thoroughly as any engineer the insufficiency of our knowledge of the behavior of soils under the varying conditions met with in engineering, I felt that further

* Head of Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology.

investigations along this line were of great importance, and that it would be well worth while for Technology to invite Dr. Terzaghi to come to the Institute for a year as a Lecturer and Research Associate in order that our staff and students might become familiar with his work and co-operate with him in carrying it forward. I think the large audience here this evening clearly shows the desire of engineers to gain further knowledge concerning this important subject, and I believe that all of you will carry away with you a greater appreciation of the problems confronting us and of the methods of attack necessary for their solution than you had when you came.

MR. LAZARUS WHITE: * This lecture appealed to me so much that I think if a second one were to be given in Constantinople I should go there to hear it also. I rather envy the Boston engineers their opportunity to hear Dr. Terzaghi lecture more frequently. We hope to hear him in New York, also, in lectures similar to this, if it can be arranged.

I was struck by some articles by Dr. Terzaghi which have appeared recently in "Engineering News-Record," and by the great value of his observations and experiments in explaining results we have seen and have not been able to explain. We saw these things and were rather dumbfounded by them. I was brought up in the old classical school of "Rankine." Rankine was the engineer's Bible. After graduating I was put underground, and I have remained underground ever since. It wasn't long before I found that the classical theories didn't apply at all. Contractors with no knowledge of Rankine were doing things that were utterly impossible if one really believed in Rankine. After being an engineer for fifteen years or so I became a contractor and started doing those impossible things myself. They were impossible to Rankine, but they are not to Dr. Terzaghi. I have been following this branch of engineering for a long time, and have found personally that his theories are justified. We have been doing as many as ten jobs simultaneously and sixty or more a year, so we have had ample opportunity to observe and check up his theories. We also have done some experimental work in a cruder way but on a larger scale.

To show this I have brought with me a set of slides. Some of our data were obtained at as early a date as ten years before we heard about Dr. Terzaghi.

Figs. 19 and 20 present the results of loading tests made by means of a hydraulic equipment, — a block, a jack and a gauge. The curves become steeper with increasing load. Dr. Terzaghi was able to explain that theoretically. Fig. 19 shows the arrangement used for making

* President, Spencer, White & Prentiss, New York City.

loading tests next to existing buildings, for underpinning purposes. The horizontal beam is braced against the building by a strut, and with the hydraulic jack we exerted the pressure required. Fig 21 is a photo-

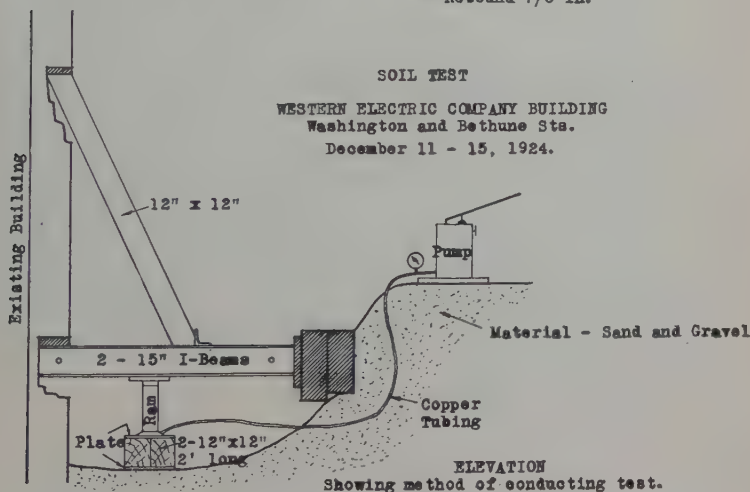
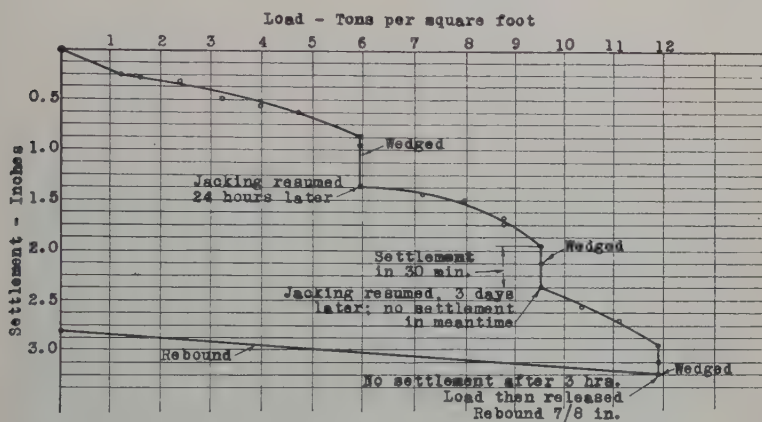


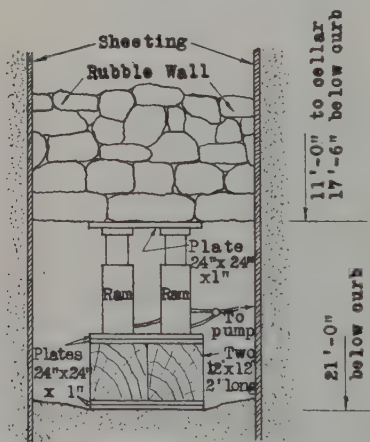
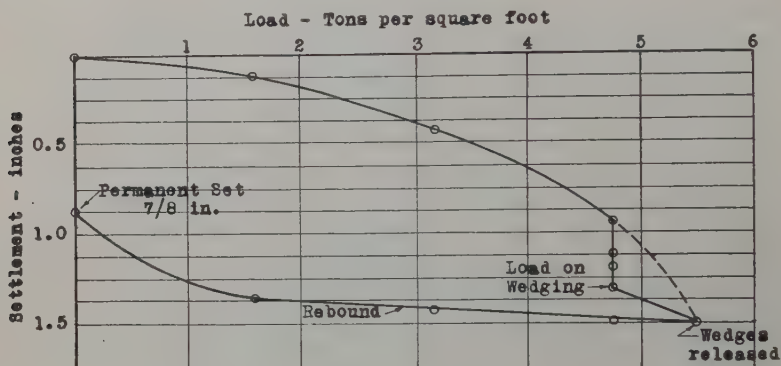
FIG. 19

graphic view of the test arrangement. The settlement curve, Fig. 20, gives striking evidence for the elastic "rebound" of the soil.

If there is no wall next to the pile against which to brace our hydraulic jack device, we build a platform entirely clear of the pile we want to test. After loading the platform with 100 tons of pig iron we insert a hydraulic jack and run up our load. In order to keep the pile

under pressure for a longer period of time, we insert a beam between the jacks. We call this process the "Pretest System."

The results of these and many other tests we have made are in full agreement with Dr. Terzaghi's theories.



SOIL TEST - PRETEST METHOD

83 John St., N. Y.

Made in connection with
Jos. Stewart & Co.

April 17, 18, 20, - 1925

Material: Fine Sand with
a trace of Clay.

Size of Rams: 4.5 in. Diam.
Area - 15.9 sq. in.

FIG. 20

ALLEN HAZEN: * I consider myself very fortunate to be with you tonight. Some years ago in studying hydraulic fill dams in California, many of these matters were necessarily considered, and it certainly would have been useful to me if I could have had the suggestions then that were presented to us tonight.

In the California work we came to the conclusion that the most useful index of stability in a given line of materials was the percentage

* Consulting Engineer, New York.

of voids. A great deal of attention was given to methods of ascertaining voids rapidly and with comparative accuracy, and in a general way it seemed to be true that materials, whatever their grain size that had become compacted to a point where the voids were 40 per cent or less, were so stable that there was not much to worry about in regard to them. The materials that occasioned the slip usually had 50 per cent or more of voids. Intermediate materials with from 40 to 50 per cent of voids were in a doubtful class and required further study.



FIG. 21. — Pretest soil test at Washington and Bethune Streets, New York City

In another matter the author has given me comfort. In the hydraulic fill dam work I laid down the rule that for a material to solidify promptly down to the 40 per cent voids class it was necessary that it should not contain very many particles less than 0.01 mm. in diameter. If the material contained many particles finer than that it would settle so loosely as not to come into the stable class within a reasonable length of time. The author's admirable illustrations of tubes in which equal amounts of materials of varying grain size, but otherwise of the same character, had been allowed to settle in water, gives a graphic demon-

stration that his results are in substantial accord with the rough basis which was then reached.

I congratulate the Massachusetts Institute of Technology on having borrowed Dr. Terzaghi as a lecturer.

MR. GEORGE PAASWELL: * I came over from New York to listen and not to talk. I have been familiar with Dr. Terzaghi's work for five or six years, and have sensed the growing importance of his discoveries, but I never dreamt that I should actually hear him speak of them before an American audience of engineers. The whole science of soil mechanics is as yet so embryonic, and the whole practice of earth work construction has seemed so controlled by superstition, that until recent date it has appeared to be hopeless to expect that this new science would make any impression upon usual practice. For this reason, the step taken by the Institute, in bringing the Doctor here, both to teach and to experiment, is freighted with unusual interest, even though it seems a trifle hard to make us wait for the next generation to bring his teachings into active practice. One point the speaker would like to stress above all: in spite of sporadic experiments carried on by others, it is upon the shoulders of Dr. Terzaghi alone that the new science of soil mechanics is being carried, and that seems to be a very unequal burden to place upon one man. A vast amount of experimenting may be carried on by others, even though we must depend upon his genius to make those keen interpretations of the phenomena attendant upon soil work, as well as to lay out plans of testing apparatus. For this reason, the speaker, as a member of the Soils Committee of the American Society of Civil Engineers, would appreciate every helpful bit of data culled either from practice or from laboratory work. Every one may co-operate, and the speaker urges that all look upon this new science with an open mind. There is nothing mysterious in what Dr. Terzaghi has said. He is not pulling rabbits out of a silk hat. One may perform the same experiments that he has, using the same brilliant methods of attack, and substantiate the astounding theories put forth by him. From the enthusiasm that has greeted his work, I cannot help but feel that in a comparatively short time we shall see some of his theorems put into the earth-work code of actual practice.

MR. FREDERIC H. FAY: † It has been certainly most enlightening to all of us to thus gain some insight into the scientific basis of soil mechanics. One thing which impressed me is that in our ordinary methods of judging soils by sieve testing we do not get at the root of

* Consulting Engineer, New York.

† Of Fay, Spofford & Thorndike, Boston.

the matter. The materials which Dr. Terzaghi has been discussing tonight are very much finer than anything one deals with when making an ordinary physical analysis. In other words, Dr. Terzaghi begins where our sieves leave off, and he studies the behavior of the more minute particles. We are only at the threshold of the science of soil mechanics, — the substitution of rational scientific basis for guesswork and rule-of-thumb methods in which we form our judgments according to the past experiences of ourselves and others. Sometimes we can predict results with reasonable accuracy, but we do not know why things happen. A previous speaker has well said that Dr. Terzaghi is perhaps today the outstanding figure in this new field of earth science. In these fundamental investigations he has been working almost alone. It is a heavy burden for one man to carry, and the burden should be distributed. Could not Dr. Terzaghi best aid his own work and the engineering profession by giving the widest publicity to what he has accomplished and to what appears to him to be the next most important steps — suggesting and outlining experiments to be carried on by others, so that other engineers, like Mr. White of New York, who are working along these lines, may work more effectively, and the efforts of many may be better co-ordinated for the solution of these most important problems? Well-directed effort by many toward the common end will mean a more rapid advance of the science of soil mechanics, and thus we may not have to wait a generation or depend wholly on Technology for results. Perhaps Dr. Terzaghi will offer suggestions for tests and experiments in which others may co-operate toward the practical solution of problems common to all engineers whose work brings them in contact with the soil.

DR. TERZAGHI: I wish to thank Professor Spofford for his words of appreciation. The attendance of the course on soil mechanics exceeds what has been expected, and seems to justify Professor Spofford's educational experiment.

The records presented by Mr. Lazarus White are unusually interesting. Mr. White has probably more experience in the difficult field of underpinning than any other contractor in the world, and the statement, that the results of his observations concerning the elastic behavior of the ground confirm my theoretical conceptions, gives me great satisfaction. Laboratory tests merely have the purpose of guiding our mind in its search for the physical causes of the phenomena. However, for confirming our conclusions, for applying our theoretical insight to the problems of actual practice, and for broadening the empirical basis of our science, we urgently need the results of such tests as

those described by Mr. White. For this reason, the material accumulated in Mr. White's office represents a precious capital. I wish to thank Mr. White for the opportunity to see some of his valuable records, and for the kindness of visiting us in Boston.

I highly appreciate the privilege of having Mr. Allen Hazen among the audience. His investigations concerning the effective size of the core materials of hydraulic fill dams undoubtedly present the most important event in the short history of the hydraulic fill method. Prior to the publication of Mr. Hazen's results, hydraulic sluicing was a gamble. His results are in perfect agreement with what I have observed in the laboratory since. The only one point which seems to require further investigations, and where rather important new results ought to be expected, concerns the shape of the grains. The coefficient of permeability of a sand or silt with bulky grains was found to be many times greater than the coefficient of permeability of a sand or silt with the same effective size and the same uniformity coefficient, but with an appreciable percentage of mica as a constituent. According to the thermodynamic analogy mentioned in my paper, the speed of consolidation of a fine-grained deposit essentially depends on the coefficient of permeability of the core material, comparable to how the rate of cooling of a hot body of a given shape essentially depends on the coefficient of heat conductivity. Hence, the shape factor is expected to have a rather important bearing on the rate of consolidation and the degree of stability of sluiced cores, consisting of materials with equal uniformity and equal effective size.

The presence at this meeting of Mr. George Paaswell is for me a very joyful event. During the preceding years of single-handed struggle with tough problems and with an ultra-conservative public opinion, he has been one of the few engineers who anticipated the latent possibilities of my methods of attack. For this reason I consider his being connected with the Soils Committee a factor of great importance, and I trust his influence, combined with his expertship, will greatly help to establish closer co-operation between laboratory and field.

The suggestions made by Mr. Fay certainly are most welcome, since nothing seems to me so desirable than associating my own efforts with those of others.

Co-operation with the laboratories of other institutions of learning is seriously handicapped by the fact that all of my original publications are in German. Successful continuation of my laboratory work requires, above all, intimate acquaintance with the theoretical principles laid down in my works.

However, no such obstacle prevents active co-operation with the engineers working in the field. Most valuable results could thus be obtained, and it gives me pleasure to specify some of the data we need. They can be grouped as follows:

- A. Bearing Capacity of the Ground.
- B. Pile Foundations.
- C. Landslides.
- D. Quicksand Phenomena.
- E. Quantity of Water Flowing into Foundation Pits.
- F. Cofferdams.
- G. Earthdams.

Group A. Bearing Capacity of the Ground. — When making test borings, undisturbed samples should be extracted from the drillhole by means of a sampler, whenever the ground is such that it can be done (coherent soil). Such undisturbed samples, *i.e.*, samples with their original water-content, should be transferred at once into a glass container, which fits the sample with a minimum air space between sample and container. The container should be sealed with wax or paraffine. In such a container, the sample retains its original water-content during several years, provided there is no appreciable air space left within the glass. No such precautions are needed, in keeping samples of sand or non-coherent silt. Samples washed out of the hole and collected in a bucket are worthless. Some typical samples should be kept for at least two years, because in case the engineer intends later on to publish his construction data it is highly important to have these samples tested in an engineering soils-laboratory of the kind M. I. T. plans to establish, and to incorporate the results in the published report. In addition the publication ought invariably to contain a cross-section representing the results of the test-borings.

Loading tests of the ground always are highly desirable, in spite of our uncertainty concerning the interpretation of the test results. Each published test-record combined with the record of test-borings and the results of subsequent settlement observations represents one step towards increasing our capacity to forecast the settlements.

Loading tests ought to be made on an area of at least 2 square feet, located on the bottom of a test pit at least 5 feet deep, 5 feet long and 5 feet wide. If the groundwater level is within 7 feet of the surface, excavation should be carried down to the water level. On the other hand, if the loaded area is located above the ground water level, the site ought to be protected against rain by a temporary roof.

For making the loading tests, the apparatus proposed by the Soils Committee of the American Society of Civil Engineers * could be used. If the soil has but little cohesion, the protective sandlayer provided in the Soils Committee's design should be omitted. I preferred to use for my loading tests the home-made device shown in Fig. 22, because I found it cheaper and sufficiently accurate. The load is carried by a rectangular wooden frame. To prevent horizontal motion, one end of the frame is supported by a simple sawhorse; the other one rests on a T-shaped frame, whose lower end rests on the ground that is to be tested.

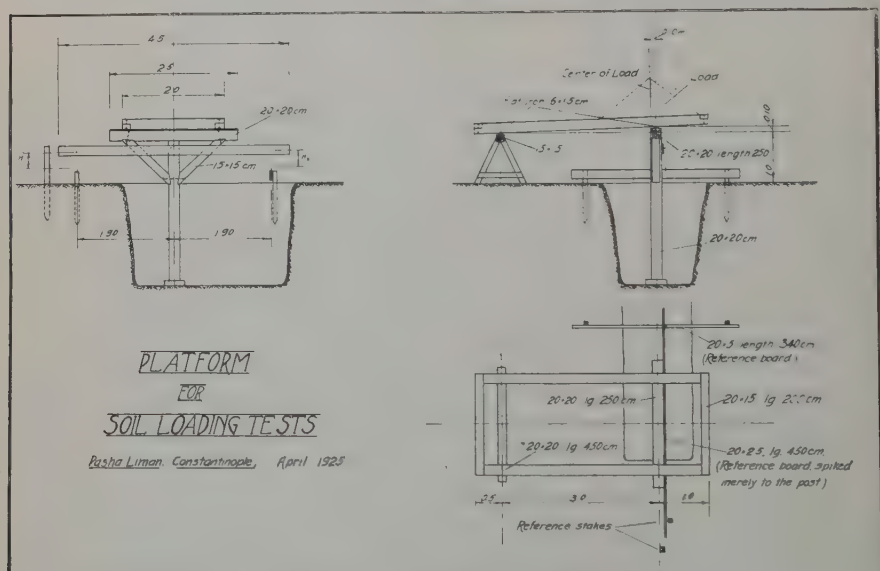


FIG. 22

The load is applied with an eccentricity of about 4 inches, a fact which has to be taken care of when computing the pressures. In the sketch, Fig. 22, the dimensions are given in the metric system. The soil pressure due to the weight of the T-frame and the top frame must be known. Pointed nails projecting out of the heads of two reference-stakes serve as bench marks, and the readings ought to be made with an accuracy of about $\frac{1}{16}$ inch. If the crosspiece tends to dip towards one side, the next load-increment should be applied such as to bring the crosspiece back into its horizontal position. As a load we used rails, I-beams or reinforcing bars, each load-increment being weighed on a platform-scale be-

* Papers and Discussions, August, 1920.

fore being applied. Particular accuracy is required while making the readings at low load. After having applied half of the total load, a pause should be made of twenty-four hours, during which hourly readings should be taken throughout the daytime. A similar set of time-observations, covering a period of at least twenty-four hours, should be obtained after full load is applied. If making loading tests on saturated silt or plastic clay, the pause ought to be extended over at least four days, until the increase in settlement at constant load becomes practically negligible. When removing the load, observe the rebound at two-third, one-third and zero load, respectively.

One of the most important parts of the work consists in establishing several bench marks at accessible parts of the foundations of the building. Precise levels should be run over these bench marks during construction, and at least two such levels during the first two years after the building was finished. Besides a permanent reference bench mark, the levels ought to include at least one point located more than 100 feet away from the building in order to make sure that the settlement of the ground is confined to the area loaded by the building. The report on the results of the levels should include diagrams showing the dead-load and the live-load acting at the time when the levels were made.

Group B. Pile Foundations. — Test-boring records and settlement data as specified in Group A. Several piles should be selected for measuring the total penetration from blow to blow, starting at the first one. A second set of records to be obtained after a pause of at least two days should show the total penetration for each consecutive blow, when driving the piles 1 foot deeper. A French railroad company and the French Sociétés de Quais working in Turkey make it a rule for keeping such records for every pile driven by the Company. Such data seem superfluous. It is preferable to select only a few piles, and to keep the records carefully.

The records for the test piles should contain: type of pile-driving outfit; weight of hammer; length, upper and lower cross-section and approximate weight of the pile; type of cap used for protecting the pile-head; height of the fall of the hammer each time; and the total penetration after each blow. Similar data are required for test piles, driven by steam hammer. Supplementary data for all piles: spacing of piles, order in which the piles were driven, level of the ground before driving, and level of the ground after all the piles are in. (Important, because it gives some information concerning the increase in density of the ground, produced by driving the piles.)

LOADING TEST ON PILE N° IV

Basha Liman, Constantinople, April 1925

DETAILS OF LOADING PLATFORM

Weight Reinforcing Steel. Total 45,512 tons

On account of the eccentricity of the surcharge, the load has to be reduced by 4.8%

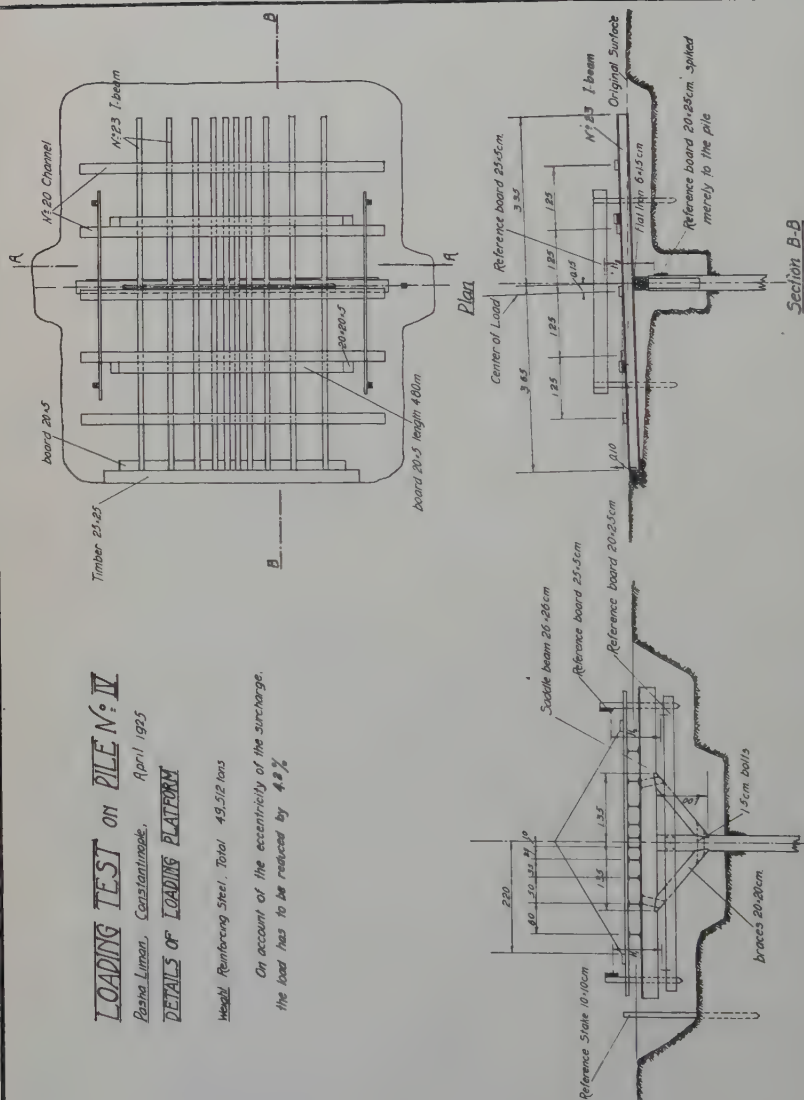


FIG. 23

If driving reinforced concrete piles, a considerable part of the impact is taken up by the protective elastic cushion, resting on the pile-head. For this reason, conclusions drawn from penetration-observations with reinforced concrete piles are not reliable. Hence the observations mentioned should be made with fairly cylindrical wooden test piles, if there is any possibility to do so. The error due to difference in cross-section (circular and square) and difference in skin friction (friction on timber and friction on concrete) was found to be considerably smaller than the error due to the elasticity of the pile-cap.

Pile loading tests have the bad reputation of being very expensive. By using a device of the kind shown in Fig. 23 I succeeded in making such tests at very low cost. The loading platform consists of a set of I-beams, rails, or square timber, or any other kind of available beams properly spaced but not connected with each other. To prevent horizontal movement, one end of each beam is supported by a board resting on the ground and bears against the face of a square timber. At the center point the beams are supported by a crosspiece that bears on and is braced to the pile. The load is carried by a few cross timbers (rails or channels) placed at right angles to the platform-beams. In order to maintain the stability of the system, the center of gravity of the load is 6 inches off the center of the pile towards the supported end, which fact must be considered when calculating the loads, acting on the pile. For making the loading test, the rules for using the device shown in Fig. 22 apply, without any modification (see loading tests, Group A). It is self-evident that the reference-board must be spiked to the pile only, and not to the braces.

In making a loading test and using the device shown in Fig. 23, the amount of labor required to load a pile up to 50 tons with round, reinforcing bars, after the site is prepared, and to observe the rebound produced by removing the load, was as follows (supervision excluded):

Making and removing the platform: 3 carpenter days; 6 laborer days.

Applying and removing the load: 16 laborer days.

No special material should be required, as both the platform and the load can be obtained from the construction material almost without any loss due to cutting. The cost of the tests are so low that they are always justified, even in connection with construction work of minor importance. If using any one of the common pile-driving formulæ, one is obliged to operate with a factor of safety of 6, on account of the uncertainty of the values furnished by the formulæ. With the results of a loading test at disposal, one can reduce the factor of safety to 4 or 3.

As a consequence, the economy thus realized more than justifies the costs of the test.

The piles should always be loaded until the penetration amounts to at least 4 inches, because the shape of the settlement-curve at high loads is very variable for different materials and most characteristic for each one of them.

After the loading tests are completed, a pulling test is very desirable. Several devices are used for making pulling tests. One of the simplest consists of a long lever, made out of rails, I-beams, or of square timbers. One end of the lever is attached to the head of the pile which should be pulled, the other end carries the load. The middle support of the lever consists of a short piece of rail resting on the head of one of the piles located next to the test pile. The ratio between the shorter and the longer lever arm should be at least 1:5. In Europe such devices have been repeatedly used. The report on the result of the pulling test should contain the total force required for pulling the pile, the dimensions of the pile, the area of contact between the pile and the ground, and a statement, whether or not the ground located around the pile was visibly forced up while pulling the pile.

Group C. Landslides. — Careful investigation of the conditions which preceded the accident (drainage conditions on the surface before the slide occurred; shrinkage and settlement cracks; weather conditions, particularly rainfall, at and before the time of the accident, etc.). Samples with their original water-content of the material forming the steep slope in the back-ground of the disturbed area, and similar samples of the material which went down, at least 3 pounds of each, should be kept for testing in some soils-laboratory. Concerning the method of preserving the samples, consult Group A (test-boring samples).

Contour plan and cross-sections of the ground before and after the slide. Sketch showing the shape and the dimensions of the disturbed area, report on evidences of pre-existing sliding planes, visible sliding planes, highly permeable intermediate layers. Results of test borings, if such were made.

Group D. Quicksand Phenomena. — General plan of the situation. (Dimensions of excavation pit, elevation of ground-water level, type of excavation and timbering, type of sheet piles, method of driving sheet piles, depth to which the piles were driven, type, capacity and location of pumping machinery (point of suction). Results of test borings as specified in Group A.) Description of the observed phenomena. A 5-pound sample of the quicksand to be kept in bottle or in bag. If possible: Determine the volume of voids or the actual water-content of

the *undisturbed* ground. No data concerning this important factor are available thus far. In order to determine the volume of voids of the undisturbed quicksand, level off an area of three or four square feet of the quicksand deposit. Take a 10-inch section of a sheet-iron or galvanized-iron pipe with a diameter of about 5 inches and force it slowly into the ground by hammering the upper edge of it. The lower edge of the pipe should be sharpened, the sharp edge located at the innerface of the pipe. After having completely forced the pipe into the ground, excavate around the pipe, and finally cut the pipe with its content off the ground by means of a knife or a straight edge. Thus the pipe contains a known volume of undisturbed material. The volume of voids can be computed from the dry weight of the pipe-content, the average specific gravity of the grains and the volume of the pipe. For obtaining these data, the content of the pipe should be shipped to a soils laboratory, in a sealed tin.

Group E. Quantity of Water flowing into Foundation Pits. — Plan of the situation (as in Group D). Quantity of water entering the pit (*a*) through the enclosure and (*b*) through the bottom. The total quantity should be computed from the discharge of the pumps, and the relative importance of the increments (*a*) and (*b*) should be roughly estimated from visible evidence. Condition is that the pumps keep the water level below the bottom of the foundation pit, else no distinction could be made between the increments (*a*) and (*b*), and no conclusions could be drawn concerning the permeability of the underground. The records are particularly valuable, if the increment (*a*) is negligible and if the ground is fairly uniform. Five-pound samples of typical ground material should be kept as a record.

These observations are apt to furnish information concerning the permeability of alluvial fills. Experience has shown that certain coarse sand and gravel deposits are very permeable and others almost impermeable, depending on the percentage of fine-grained material they contain. On the other hand, the content of such deposits in fine-grained materials seems to be closely related with the geological history of the deposits. Since it is very difficult to withdraw drill samples of coarse-grained sediments without losing the finer particles, it would be useful to learn by experience the approximate limits between which the permeability of coarse river deposits may vary and to co-ordinate our experience with geological facts. The data mentioned under this heading are sufficient for roughly estimating the coefficient of permeability of the underground.

Group F. Cofferdams. — Plan of situation as Group D. Observa-

tions concerning the degree of permeability and stability of the cofferdam. If the cofferdam is of the box type, a representative 5-pound sample of the fill should be kept. Important to state how the material looked when excavated (in stiff chunks, or thoroughly wet), and how it was filled into the box and compacted.

Group G. Earthdams. — Complete construction data and the results of test borings as specified in Group A. Ten-pound samples of the earth materials finally used in the various parts of the dam should be kept. Results of subsequent observations concerning saturation line (standpipe observation) and seepage losses.

This brief review may be sufficient to show which data the practicing engineer should try to collect. It may be noticed that I made no suggestions whatsoever concerning the analysis of the soil samples. This fact expresses my opinion concerning the future relation between field and laboratory. The mere knowledge of soil physics is not sufficient for making successful soil investigations. Such investigations require much laboratory experience, and a well-equipped laboratory in addition. For this reason, I believe that the testing of soil samples should be made in permanent laboratories by experienced experimenters. For the practicing engineer the essential thing is to have a clear conception of the information the laboratory man is apt to furnish, to know how much soil material is required for soil analysis, and to know how the samples should be preserved and shipped. This unavoidable specialization prevents by no means intimate co-operation between field and laboratory. Whenever soil samples are delivered (prepaid) at M. I. T. for the purpose of securing information to be embodied in professional papers, I will always be glad to examine them free of charge as soon as I have the time required to do it. Published reports including all the data specified in my closure are so utterly scarce that every one of such reports would represent a most useful contribution to engineering science, regardless of the relative importance of the work described.

In publishing construction data it is essential to keep, above all, the following fact in mind: Hardly any report on foundation work is worth being printed unless it is supplemented, either at the time of its publication or later on, by the results of careful settlement observations, as specified above. Foundation projects merely represent our *intentions*, and the practical result is, by necessity, always more or less different. The mere intention means to us but very little. If some of the engineers present at this meeting would actually try to co-operate with me on the basis suggested, we may be in a position to establish in

New England the first example of how underground conditions ought to be investigated for engineering purposes. New England seems to me better fit for such a start than any other states I know of, for two reasons: It presents a remarkable variety of difficult underground problems, and in addition, it is remarkably well explored as far as its geology is concerned.

I highly appreciate the active interest expressed by the speakers, and I wish to thank the Society for the unique opportunity of presenting some of my results before so distinguished an audience.

OF GENERAL INTEREST

REPORT ON METROPOLITAN WATER SUPPLY

Investigating Commission Reports to Legislature — Recommends Construction of a Filtration Plant for South Sudbury System and Reservoir on Upper Ware River — Urges Immediate Appropriation of \$27,500,000

One of the most important matters which will occupy the attention of the Massachusetts Legislature at the coming session is the Report of the Metropolitan Water Supply Investigating Commission, recently submitted in accordance with chapter 491 of the Acts of 1924.

This commission after more than a year's study has recommended:

"(a) That a special construction commission be created with authority to improve and increase the present water supply of the metropolitan district and of the city of Worcester.

"(b) That the commission should immediately construct a suitable filtration plant for making available the waters of the South Sudbury system.

"(c) That the commission also should proceed immediately with the construction of a reservoir on the upper Ware River for the joint use of the city of Worcester and the metropolitan district.

"(d) That an agreement should be entered into between the city of Worcester and the Commonwealth whereby the latter shall construct the said reservoir and its necessary appurtenances, the city contributing one-ninth of the actual cost of same in return for the right to take one-ninth of all the water thereby made available, with the further privi-

lege of taking from time to time in the future additional amounts of water when needed, in increments of one-ninth each, paying therefor one-ninth of the original cost for each one-ninth of the supply of water so taken, and in addition contributing to the annual cost of maintenance and upkeep in such proportion as its taking of water bears to the capacity of the entire supply.

"(e) That the commission be authorized to purchase land at a suitable point along the Weston Aqueduct for the location of a future filtration plant of sufficient capacity to filter the amount of water capable of being carried by said aqueduct.

"(f) That the commission be empowered to purchase or take by right of eminent domain such land, rights, easements or other property within the watersheds of the Assabet and Ipswich rivers as it may determine to be necessary for the proper protection of those sources against future encroachment by private interests and for preserving the sanitary quality of their waters.

"(g) That the commission be instructed to study and to report their recommendations to the General Court on or before December 1, 1926, as to the need, location and estimated cost of delivering the future water supply to the distribution system of the metro-

politan district either by pressure tunnel or by pipe lines.

"(h) That the Legislature appropriate the sum of \$27,500,000, and that proper authority be given the said commission for expending the same in carrying out the foregoing recommendations."

The Metropolitan Water Supply Investigating Commission, appointed by the Governor on August 20, 1924, consisted of Chas. R. Gow of Boston, chairman, E. E. Lochridge of Springfield, and George F. Booth of Worcester. Allen Hazen, well-known engineer of New York, served as consulting engineer, and Raymond C. Allen of Manchester, Mass., acted as secretary of the commission in addition to other duties of an engineering nature. Mr. Hazen conferred frequently with Leonard Metcalf of Boston on various matters in connection with the report. It is interesting to note that all of these men, with the exception of Mr. Booth who is not an engineer, are active members of the Boston Society of Civil Engineers.

The engineering work consisted of a complete review of all previous studies, particularly the report of the so-called Joint Board — the State Department of Health and the Metropolitan Water and Sewerage Board (the latter since merged in the Metropolitan District Commission); together with an investigation of every available new source of supply which appeared at all promising. Considerable surveying was required and borings taken in connection with the several sources and projects considered. A detailed engineering report by Mr. Hazen accompanied the commission's report.

An important part of the work of this commission was the study of such additional sources of supply for the city of Worcester as would best tie in with any project proposed for increasing the supply for the metropolitan district.

The following excerpt from the re-

port outlines the availability and estimated costs of developing the various sources of water supply considered by the commission:

"(1) The present sources of water supply of *satisfactory quality*, available for the needs of the metropolitan district, can be depended upon to furnish an average output of not more than 119,000,000 gallons per day during a series of exceptionally dry years such as frequently occur, whereas the district is now consuming approximately 130,000,000 gallons per day and is called upon for additional amounts for other communities, with the probability that an even greater total amount will be needed in the immediate future.

"(2) In addition to this amount of good water there is also available an existing supply of some 26,000,000 gallons per day obtainable from the South Sudbury system, the present natural quality of which is so unsatisfactory that for many years past it has been used only in extreme emergencies and for short periods when there was no other alternative. This supply can be made equally satisfactory, or better than that now used, by constructing suitable purification works at an expenditure of approximately \$3,500,000 which will thereafter assure to the district a total safe capacity of 145,000,000 gallons per day, sufficient, according to our estimates, to meet the probable water needs of the district until about the year 1929. Before that date, therefore, there must be developed some new supply if the danger of a probable water shortage is to be avoided, and the period required for constructing such works will probably be not less than four years under the most favorable circumstances.

"(3) The city of Worcester is in an even more critical state respecting its water supply than is the metropolitan district, since it is already experiencing an actual shortage of supply and has been compelled to purchase from 5,000,000 to 6,000,000 gallons per day from the metropolitan district for several months past and to pump the same from

the Wachusett Reservoir into its own system. This water used by Worcester is in addition to the 130,000,000 gallons per day now being consumed by the district itself.

"(4) Additional water, suitable for the future needs of the metropolitan district and of Worcester, may be secured by developing the Ware and Swift rivers for that purpose, as originally suggested as a future possibility at the time the metropolitan system was created, and as recommended still more recently by the Joint Board, so called, which reported upon the subject in the year 1922, or otherwise various smaller developments may be utilized which can be added to the system from time to time in the future as the demand for greater quantities of water occurs.

"(5) The Ware and Swift river project as recommended by the Joint Board would, if carried out, more than double the present capacity of the existing supply, including that proposed to be reclaimed by filtering the South Sudbury water, and would care for the combined needs of the metropolitan district and of Worcester for perhaps as much as seventy-five years into the future. This source would yield a water of excellent natural quality, and its cost *per million gallons of capacity* might be as low as that of any other source or sources which can be suggested as substitutes. The entire investment, however, would have to be made substantially at once for an amount of water which would not be required for many years hence or possibly never if by any chance the growth in population were for any reason to cease or decline.

"(6) The taking by the metropolitan district of the waters of the Swift and Ware rivers, together with the large areas of land required for reservoirs and protective measures, is strongly objected to by the communities in that section of the state because of what is deemed to be an unnecessary encroachment upon existing local rights and benefits pertaining to these natural resources. Also, the proposed method of utilizing this source would not meet in a satisfactory manner the requirements of the

city of Worcester for an addition to its present supply, since all of the water which that city might secure from such a development would have to be elevated several hundred feet by pumping, and its taking would necessarily be subject to such restrictions and regulations as might be imposed by the metropolitan district.

"(7) There is, however, a possible solution of Worcester's water supply problem which would seem to be advantageous alike to that city and to the metropolitan district. By constructing a dam across the Ware River in the vicinity of Barre Falls a large reservoir would be created in the valley above that point which would have a capacity of 45,000,000 gallons per day, which could be made to flow by gravity either to some of the existing reservoirs of the Worcester system, to the Wachusett Reservoir, or to both. Since the amount of water which would be made available by this means would be considerably in excess of that required by the city of Worcester for a very long period, there would remain during that time a substantial surplus which could be utilized by the metropolitan district to augment its present supply and thus to defer to the occasion when additional sources would be required for the enlargement of its own system. The estimated cost of developing this supply is \$14,000,000.

"(8) In addition to the supply obtainable from the North Ware, it will be possible, whenever it is needed, to secure 47,000,000 gallons per day of additional supply by diverting certain branches of the upper Assabet River into the Wachusett Aqueduct which passes through the watershed of that river on its way to Sudbury Reservoir. The cost of such a development will be approximately \$8,500,000.

"(9) When the time comes for securing more water than these two sources are capable of delivering, the Ipswich River above the town of Topsfield can be developed as a third source of supply capable of yielding 80,000,000 gallons per day, 30,000,000 of which may ultimately be required for the water needs of communities in or adjacent to the

Ipswich valley. The remaining 50,000,000 gallons per day could be pumped to the metropolitan system in the vicinity of Spot Pond. The total cost of this development at present prices would be in the neighborhood of \$19,000,000.

"(10) The Hobbs Brook Reservoir of the city of Cambridge water supply can be greatly increased in size by raising the present dam some 30 feet. If this were done it could be kept full by pumping water from the lower Sudbury River during seasons of flood, and by such means the capacity of the supply might be increased 50,000,000 gallons per day. The cost of this development would be about \$12,000,000.

"(11) The total additional supply which can ultimately be secured by developing successively the North Ware, Assabet, Ipswich and Hobbs Brook sources will be 202,000,000 gallons per day, and the aggregate cost approximately \$53,500,000. These figures compare with approximately 200,000,000 gallons per day obtainable from the Swift, Ware and Millers rivers at a cost as estimated by the Joint Board in its 1920 report of \$60,000,000.

"(12) The water obtained from the Assabet, Ipswich and Hobbs Brook sources would require filtration before using, while that of the North Ware supply, if used in considerable quantity, might tend to lower slightly the present quality of the Wachusett water with which it would necessarily mix in passing through the system. On the other hand, unless very considerable sums are expended in the near future in the purchase of property adjacent to the Wachusett Reservoir, for the preservation of the purity of the supply, it is doubtful if resort to filtration of that water can be much longer deferred.

"(13) The Quinebaug River has also been considered as a possible source of future supply. Considerable quantities of water of excellent quality might be obtained from this source at a cost comparable with other supplies studied, but since sufficient water is obtainable from nearer sources, no recommendation is made at this time with respect to this supply.

"(14) Aside from the matter of securing some new source or sources of supply, there is also involved the important question of bringing the additional water thus obtained into the metropolitan district. Existing demands require the full capacity of available conduits from the Weston terminal of the Weston Aqueduct to the centers of distribution. Certain engineering studies which have been made in the course of this investigation indicate the desirability of constructing a deep pressure tunnel in the underlying rock between the Weston terminal and a point in or near the city of Everett. The estimated cost of such a tunnel is \$17,000,000. In addition, some present deficiencies of capacity in certain sections of the Weston Aqueduct itself now require correction at an estimated expense of \$700,000."

The report, which will be available later in printed form when under consideration by the Legislature, contains a great deal of interesting information concerning the history of the metropolitan water supply, capacity of existing sources, probable life of present supply, and possible additions of municipalities to the metropolitan water district. The following table is given showing the estimated Future Population and Water Consumption of the Metropolitan District, including Newton and Brookline, together with fifteen surrounding municipalities and the city of Worcester:

	Estimated Population	Estimated Consumption (m. g. d.)
1925	1,890,456	182,000,000
1930	2,020,000	202,000,000
1935	2,150,000	221,000,000
1940	2,270,000	240,000,000
1945	2,390,000	260,000,000
1950	2,510,000	280,000,000
1955	2,625,000	302,000,000
1960	2,740,000	325,000,000
1965	2,850,000	350,000,000
1970	2,960,000	375,000,000

The aggregate capacity of the combined sources of supply now available

for the needs of these several municipalities is as follows: —

	m. g. d.
Present metropolitan supply . . .	145
Newton and Brookline supplies . .	10
City of Worcester supply	20
Fifteen surrounding cities and towns	23
	198

Careful consideration is given to the report of the Joint Board, previously referred to, and after summarizing the advantages and disadvantages of the plans proposed in that report, the present commission states why its own alternative plan is preferred to that of the Joint Board.

Much attention is given to the sources of supply of Worcester, its rights in the Wachusett watershed, and the proposed plan for increasing the supply for Worcester through the joint use by that city and the metropolitan district of the North Ware additional supply.

The deficiency of the present distribution system is discussed and it is suggested that the special construction commission, the appointment of which is recommended in this report, should be directed to study "the need, location and estimated cost of delivering the future water supply to the distribution system of the metropolitan district either by pressure tunnel or by pipe lines."

The final and a very important part of the report is devoted to "The Financial Program Involved with an Increased Supply." Here the probable expenditures over the next thirty years under this plan are analyzed and compared with the expenditures involved if the recommendations of the Joint Board were the ones adopted. In a final comparison the report states:

"The difference in cost between the two plans may not appear to be impressive when measured in cost per million

gallons, since the excess cost of the Joint Board plan over that now proposed averages only \$6 per million gallons for the next thirty years. The total saving represented during this period, however, is approximately \$12,500,000, of which \$11,000,000 applies to the first twenty years, so that the saving to the communities affected will be very substantial. In addition to this fact the community will be supplied under the program now proposed with filtered water of the highest degree of purity as compared with natural waters which, however excellent their quality may be, are always subject to the possibility of accidental pollution and at times to tastes and odors of an objectionable nature."

In conclusion the commission reports as follows:

"As a result of a study extending over more than a year we are convinced that the metropolitan district is confronted with a critical situation respecting its water supply both from the standpoint of capacity and of distribution facilities.

"The system is underdeveloped for the service it must supply in the very near future. In our opinion immediate action is necessary in order to avoid the possibility of consequences of a serious nature. Undoubtedly the restrictions imposed by the war period and the subsequent high costs of construction have been largely responsible for the failure to keep pace with growing requirements, but further postponement will be extremely dangerous.

"In periods of sufficient rainfall it is difficult for communities to realize the possible consequences of a recurring dry spell such as is to be expected at intervals of from fifteen to twenty years. The last period of severe drought occurred about fifteen years ago. It is reasonable to anticipate another within the next few years. The time to prepare for such an emergency is before its occurrence. To wait until the crisis is here will be futile since there will not then be sufficient time available in which to provide the necessary remedial measures.

"It may be urged that there is now sufficient water obtainable with which to meet such an emergency if quality is disregarded, and this may be true to a certain extent. Its use, however, in an unpurified state will be fraught with danger to the health of the community unless chemical treatment is resorted to, in which case it is likely to prove decidedly unpleasant as a beverage and quite unacceptable to a population that has become accustomed to a high standard of quality in its water supply.

"The distribution system, particularly that portion pertaining to the delivery pipes from the Weston Aqueduct, has been permitted to lag behind the actual requirements for capacity, so that it is only by an ingenious manipulation of the pumping equipment that sufficient pressure is maintained at all times in the system. This constitutes a serious hazard in case of any exceptional demand for water, such as a serious conflagration or severe cold spell. Likewise the crippling of one of the pumps or the breaking of a force main would be likely to result disastrously. In our opinion this feature also should have immediate attention and correction, and the system should be brought at once to a state where maximum and emergency requirements can be met with a reasonable margin of safety."

Boston's Earthquake Hazard

This year has witnessed several earthquakes of minor intensities in New England. It has been somewhat disconcerting to folks who have grown up in the faith that the troubles of California need never be anticipated on our stern and rock-bound coast.

However, since the coming of the white man to New England, this area has a record of 300 quakes, varying from disturbances of minor intensities to the very severe earthquake which occurred in Boston in 1755, the recurrence of

which today would cause great loss in life and property.

The geologists tell us that the earth's crust locally is in one of its periods of readjustment to strain. We are also told that Boston is at one end of a fault or point of weakness in the earth's crust where the effects are localized, and that judging from past experience we may expect the period of readjustment to continue for the next ten years.

If the geologists are correct in their premises we may expect recurrence of earthquake shocks during the next decade. No attempt is made by the scientists to predict quakes in the sense of time and place. It is, however, stated by one geologist that sometime, somewhere, in New England, during the next decade, there is an expectancy of a quake of the severity of Boston, 1755, or Santa Barbara, 1925.

What, then, should be the attitude of Boston engineers when earthquakes are mentioned? The insurance companies report the sale of earthquake insurance in considerable amounts. Should the engineer adopt the attitude of the forefathers during our early quakes, who said they were acts of Divine Providence, and who sought to remove the cause by propitiatory supplications addressed to an angry Deity, or should he adopt the spirit of deprecation of the danger which is to be noted in California?

Is it not better engineering to recognize the danger which is inherent in even minor quakes when they occur in a congested city, with a large foreign population which is easily made panic-stricken; and also to recognize the more serious dangers during severer quakes from falling copings, veneers, and canopies, and the dislocation of water mains in filled land?

The report of a committee of engineers on the San Francisco quake assured us that well-constructed buildings of all classes withstood the earthquake well.

Flimsily constructed buildings on poor foundations suffered severely. The damage from the fire hazard was another matter and not strictly a question of the purely structural design of the building. The committee emphasized the value of an adequate structural framework with good joints.

The engineer, as a good citizen, may well stand back of any movement tending to educate the masses to take proper steps in times of earthquake emergency. Just what form such a movement should take requires very careful study, as the very thought of panic begets panic in the excitable mind.*

Whether or not Boston really has a serious earthquake hazard, local engineers may be sure that they are proceeding in the proper direction if they continue to insist on sound structural design. All engineers want buildings adequate for any reasonable loads. The continuation, on the part of the engineer in the office and field, of insistence on good design and workmanship will help in promoting the construction of buildings that may be safely expected to withstand quakes of the maximum intensity that may occur locally. Records of local quakes do not warrant the expectancy of catastrophic earthquake disturbances, and in regions where they do occur, man cannot build practical structures strong enough to withstand them.

Editor's Note: Since this article was prepared, announcement has been made of the appointment of a special committee by the Boston Chamber of Commerce to consider questions involved in the earthquake risk in Boston. This committee proposes to ascertain the general situation in Boston, and to decide what facilities are at hand to meet such a catastrophe if it should occur.

Motor Vehicles on Massachusetts' Highways

The use of the highways by motor vehicles from the inception of the automobile up to the present time was described in a most interesting manner by Mr. Day Baker at a meeting of the Highway Section, B. S. C. E., on December 3, 1925. Mr. Baker is legislative counsel for about every motor vehicle club or association in Massachusetts, and is probably one of the best posted men in the country on matters relating to legislation, regulation and taxation as they affect the owners of motor vehicles.

He traced the phenomenal growth of the automobile industry from 1895, when there were only 4 motor vehicles constructed, to 1925, when there were approximately 18,500,000 passenger cars and 2,400,000 commercial motor vehicles in use in the United States. In 1924 the number of motor busses in this country was nearly 60,000, and it is now believed that the number totals about 93,600.

Mr. Baker stated that of the 2,150,000 miles of highways in the United States in 1904, only 153,000 miles (or 7 per cent) were surfaced, whereas about 470,000 miles (or 16 per cent of the present 2,940,000 miles) are now surfaced.

The highway construction program for 1924 was, roughly, \$990,700,000, of which \$444,000,000 (or 45 per cent) was furnished by registration fees, Federal excise taxes, and gasoline taxes.

In Massachusetts the motor vehicle owners are paying for practically all of the highway construction and maintenance done by the State Department of Public Works. For the first ten months of 1925 the special motor vehicle taxes in this state netted \$9,139,000, and by the end of the year approximately

* The Engineering-Economics Foundation has offered a series of lectures at Boston recently on the Earthquake Hazard. The work was divided into three groups — insurance, engineering and general business providing space, goods or public services.

\$10,000,000 will have been received. In addition to this, motor vehicle fines amount to about half a million dollars per year.

Mr. Baker made a strong plea for wider roads, stating the 18-foot paved roadway was insufficient for the present-day traffic on important routes.

Another element that has entered into highway travel is the advent of the motor coach and bus. There are over 500 of these now in this state, operating on about 75 defined routes — some of these having been in regular operation for over ten years. Mr. Baker stated that constructive legislation is needed in Massachusetts, and that the present law is not satisfactory for the regulation of the motor busses.

A. S. T. M. Standards

During the past few months, the American Society for Testing Materials has issued two new publications. One is

a pamphlet of A. S. T. M. Standards adopted in 1925 (120 pages), containing 36 revised or newly adopted standards. This pamphlet (\$1.50) forms the first supplement to the 1924 issue of their Triennial Book of Standards. The other is the new edition of the A. S. T. M. Tentative Standards, issued annually. This volume (876 pages) contains the 194 tentative standards issued by the Society. (The price is \$7 in paper and \$8 in cloth binding.) The term "tentative standard" as distinguished from "standard" is applied to a proposed standard which is printed for one or more years with a view of eliciting criticism, of which the committee concerned will take due cognizance before recommending final action toward the adoption of such tentative standard by formal action of the Society.

The above-mentioned publications, both of which are in the library of the Boston Society of Civil Engineers, may be purchased if desired from the American Society for Testing Materials, 1315 Spruce Street, Philadelphia, Pa.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

BOSTON SOCIETY OF CIVIL ENGINEERS

NOVEMBER 18, 1925. — A regular meeting of the Boston Society of Civil Engineers was held this evening in Chipman Hall, Tremont Temple, and was called to order by the President, Richard K. Hale, at 7.30 P.M. There were about 200 members and guests present.

The minutes of the previous meeting (October 21, 1925) were approved as printed in the November JOURNAL.

The Secretary reported for the Board of Government the names of those elected to membership:

Members: Edward S. Averell,* Ralph F. Macaulay,* George E. Miers. *Juniors:* Elwood S. Avery, Waldo B. Birkmaier, Maurice Bloom, Leonard M. Hodgkins, Oscar R. Lindgren, Edward D. McLaughlin, George J. Rommer, Arlo R. Savery, George F. Schramm, Louis H. Smith, John F. Studley, Roland A. Warren.

The President announced that a two-day meeting on "Fuel and Power" was to be held by the Affiliated Technical So-

* Transferred from grade of Junior.

cieties of Boston on December 10 and 11, 1925. In accordance with a recommendation of the Board of Government, it was voted that the December meeting be omitted on account of the "Fuel and Power" meeting of the Affiliation.

It was voted that By-Law No. 1 be amended to read as follows:

"1. MEETINGS. — Regular meetings of the Society shall be held on the fourth Wednesday in January, and on the third Wednesday of the other months, excepting July and August, unless otherwise authorized by the Board of Government."

(This amendment to the By-Laws is now in effect, as it had also received an affirmative vote at the meeting of the Society on October 21, 1925.)

It was voted: "That the Board of Government be authorized to use the income from the Permanent Fund for the current year to such an extent as they deemed necessary in payment of the current expenses of the Society."

(This matter also received an affirmative vote at the meeting of the Society on October 21, 1925, and is now in effect.)

The President then introduced Dr. Charles Terzaghi, Head of the Department of Civil Engineering, Robert College, Constantinople, and Special Lecturer in Foundation Engineering at the Massachusetts Institute of Technology, who gave an illustrated talk on "Modern Conceptions Concerning Foundation Engineering." Dr. Terzaghi has had an unusually broad experience and he presented in a most interesting way the very latest developments in Foundation Engineering both here and abroad. At the conclusion of his address there was a general discussion, in which Mr. Lazarus White and Mr. George Paaswell of New York, as well as a number of members of the Society, took part.

The meeting adjourned at 9.15 P.M., after giving a rising vote of thanks to Dr. Terzaghi for his courtesy in presenting this interesting talk.

J. B. BABCOCK, *Secretary*.

DESIGNERS SECTION

NOVEMBER 11, 1925. — The regular November meeting of the Designers Sec-

tion of the Boston Society of Civil Engineers was called to order at 6.10 P.M. in the Affiliation Rooms.

Due to absence of the clerk, the minutes of the October meeting were not read.

Prof. Harry L. Bowman introduced the speakers of the evening Mr. C. R. Perry and Mr. Kendall Woodrough of J. R. Worcester & Company, who gave an illustrated talk on "The Wells River and Woodsville Bridge."

There were 50 members and visitors present. The meeting adjourned at 7.20 P.M.

SCOTT KEITH, *Clerk*.

APPLICATIONS FOR MEMBERSHIP

[December 15, 1925.]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

For Admission

CHERNESKY, ALPHONSE ALBERT, Lynn, Mass. (Age 21, b. Haverhill, Mass.) A senior at Tufts College, in civil engineering. Summer of 1925 worked in the Mass. Dept. of Public Works, Division of Highways. Refers to Robinson Abbott,

H. L. Bartlett, H. P. Burden, W. J. Downer, E. R. McCarthy, E. H. Wright.

CLAIR, MILES NELSON, Boston, Mass. (Age 25, b. Lickdale, Pa.) Graduate of Drexel Institute, 1921, with degree B.S. in engineering, and 1922-23 at Mass. Inst. Tech., with degree S.M. in civil engineering. Has had experience in charge of layout and estimates in plant construction; assistant field engineer in charge of lines, grades, etc.; as instructor at Drexel in civil engineering; at present in charge of all testing and associate in design with Thompson & Lichtner Company, Inc. Refers to J. B. Babcock, H. L. Bowman, C. M. Spofford, Hale Sutherland, S. E. Thompson.

FITTS, LELAND CHESTER, Boston, Mass. (Age 21, b. Haverhill, Mass.) Student at Northeastern University. On the co-operative plan worked five weeks with Fred Stowers, C.E., at Methuen, Mass.; in the spring of 1925 worked for Aspinwall & Lincoln and still works for them during the work periods. Refers to H. B. Alvord, C. S. Ell, J. W. Ingalls, W. E. Nightingale.

GIBBONS, MORTIMER MICHAEL, Milton, Mass. (Age 24, b. Milton, Mass.) Received S.B. degree in Sanitary Chemistry, Harvard Engineering School, 1922; chemist and bacteriologist at Poughkeepsie (N. Y.) Water Purification plant, 1922-24; 1924 to date, with Metcalf & Eddy, Boston. Refers to G. G. Bogren, S. E. Coburn, G. M. Fair, A. L. Fales.

HARMANTAS, CHRISTOS, Cambridge, Mass. (Age 23, b. Greece.) Attended public schools in Manchester, N. H. Entered Mass. Inst. Tech., 1921; graduated, 1925. Is now employed as a structural draftsman by the B. & M. R.R. Refers to J. B. Babcock, B. W. Guppy, Pusey Jones, C. M. Spofford.

HENNESSY, JOHN FRANCIS, Brookline, Mass. (Age 26, b. Brookline, Mass.) Graduated from Mass. Inst. Tech. in civil engineering in 1922. Since that date he has worked as engineer and estimator for his father, who is a contractor in Brookline. Refers to J. B. Babcock, J. W. Howard, H. A. Varney, C. J. Wallace.

PARKER, THEODORE B., Wellesley Farms, Mass. (Age 36, b. Roxbury,

Mass.) Graduate of Mass. Inst. Tech., 1911, with degree of B.S. in civil engineering. Was assistant at Tech for one year in civil engineering; draftsman for six months with H. C. Keith, consulting engineer; six years assistant hydraulic engineer with Electric Bond & Share Co. and Utah Power & Light Co.; three and one-half years in the U. S. Engineer Corps; since 1922 with Stone & Webster, Inc., in the hydraulic division. Refers to J. B. Babcock, C. S. Ell, H. A. Hageman, W. F. Uhl, D. M. Wood.

PASQUALINO, PHILIP PAUL, Wakefield, Mass. (Age 20, b. Sicily, Italy.) Studied first at Northeastern University but transferred to Tufts College in 1923, where he is now a student in civil engineering. Has had experience in construction work in Wakefield and has also worked for a private surveyor. Refers to W. J. Alcott, W. J. Downer, A. D. Fuller, E. R. McCarthy, F. B. Quinn.

THOMPSON, WILLIAM TITUS, Norwood, Mass. (Age 25, b. Fitchburg, Mass.) Graduate of Rensselaer Polytechnic Institute, 1922, with degree in chemical engineering. From September, 1922, to July, 1923, worked as sanitary inspector of summer camps, swimming pools, for State Dept. of Health, Connecticut; October, 1923, to February, 1924, with Weston & Sampson; from February, 1924, to the present, with Metcalf & Eddy, on waste disposal plant at Norwood. Refers to E. S. Chase, S. E. Coburn, A. L. Fales, A. L. Maddox.

For Transfer from Junior

JOHNSON, HENRY DEXTER, JR., Pittsborough, Pa. (Age 27, b. Marlborough, Mass.) Two-year course at Bates College, Lewiston, Me.; then entered the army, where he served one year; while in France he qualified for a 2d lieutenant's commission, which grade he now holds in the Reserve Corps; from July, 1919, to February, 1921, with the city of Auburn, Me., and the Central Maine Power Co. as labor foreman and dynamiter; in February, 1921, he entered Northeastern University. During the summer of 1921 he was foreman with the State Highway Commission of Maine; from May, 1922, to

July, 1924, was employed by the construction department of Jones & Laughlin Steel Corp., first as transitman, later as general foreman; is now construction engineer with Dwight P. Robinson & Co., Inc., on their work for the Duquesne Light Co., at Pittsburgh, in which capacity he has designed, checked and supervised the erection of a 160'-4 circuit river crossing tower for a transmission line. Refers to H. B. Alvord, C. S. Ell.

NEW MEMBERS

Members

- GEORGE E. MIERS, 346 High Street, West
Medford, Mass.
WALTER W. ZAPOLSKI, 17 Gramercy Park,
New York, N. Y.

Juniors

- ELWOOD C. AVERY, 24 East Hall, Tufts
College, Mass.
WALDO B. BIRKMAIER, 16 Harding
Avenue, Waltham, Mass.
MAURICE BLOOM, 120 Morrison Avenue,
Somerville, Mass.
EDWARD D. McLAUGHLIN, 7 Allen Place,
Melrose, Mass.
GEORGE J. ROMMER, 38 Evelyn Street,
Mattapan, Mass.
ARLO R. SAVERY, Grove Street, Silver
Lake, Mass.
GEORGE F. SCHRAMM, 21 Kittredge Street,
Roslindale, Mass.
LOUIS H. SMITH, 173 Pearl Street, Som-
erville, Mass.
HOWARD J. SPROTT, 196 Hamilton Street,
Cambridge, Mass.
ROLAND A. WARREN, 19 Queensberry
Street, Boston, Mass.

INDEX

VOLUME XII, 1925

ABBREVIATION: I. — Illustrated

Names of authors are printed in *italics*

	PAGE
Aerial Surveying	Jan., 52
Airplane to Aid in Mapping	May, 245
Annual Meeting — President's Address	April, 161
Annual Reports	April, 200
A. S. T. M. Standards	Dec., 447
<i>Barrows, Harold K.</i> Discussion — Davis Bridge Development	Jan., 33
Beams and Columns. <i>Charles E. Nichols</i>	I., Feb., 66
Beams, Bond and Anchorage. <i>Walter W. Clifford</i>	Feb., 77
Beams, Diagonal Tension and Shear. <i>Hale Sutherland</i>	I., Feb., 71
Beverly, Cast-Iron Pipe from French Foundry	March, 153
Boiler Drum built in a Gun Factory	I., Feb., 91
Bond in Footings. <i>Edwin S. Parker</i>	I., May, 246
Boston and Maine Railroad, Preservative Treatment of Ties. <i>Frank C. Shepherd</i>	I., March, 101
Boston Elevated Railway Shops at Everett	Feb., 95
Boston's Earthquake Hazard	Dec., 445
<i>Breed, Charles B.</i> Discussion — Some Problems of Highway Operation	Nov., 378, 383
Channel in Boston Harbor	May, 245
<i>Chase, Richard D.</i> Discussion — Davis Bridge Development	Jan., 44
Chimney Shells, Temperatures in Reinforced Concrete. <i>E. A. Dockstader</i>	I., March, 153
City Plan, Protecting. <i>Philip Nichols</i>	Oct., 355
<i>Clifford, Walter W.</i> Bond and Anchorage in Beams	Feb., 77
<i>Clifford, Walter W.</i> Supports for Superimposed Stacks	May, 221
<i>Coburn, Raymond W.</i> Discussion — Some Problems of Highway Operation	Nov., 384
Complex, The	Feb., 91
Concrete, Curing in a Semi-Arid Climate	Nov., 392
Concrete, Quantities of Materials	June, 303

	PAGE
Concrete. Discussion of Report of Joint Committee on Concrete and Reinforced Concrete	I., Feb., 63
Concrete, The Making of. <i>Dean Peabody, Jr.</i>	Feb., 63
Contracts for State Highway Construction	March, 153
Construction Industry, Problems of. <i>R. C. Marshall, Jr.</i>	June, 276
Construction of the Neponset Bridge. <i>J. S. Crandall</i>	I., May, 332
<i>Crandall, J. Stuart.</i> Construction of Neponset Bridge	May, 232
Culvert Pipe, Study of	Sept., 328
Curb Heights and Automobiles	March, 152
Davis Bridge Development — New England Power Company.	
<i>A. C. Eaton</i>	I., Jan., 1
<i>Delano, G. H.</i> Some Problems of Highway Operation	Nov., 365
Development of the Science of Hydraulics. <i>E. P. Hamilton</i>	I., Oct., 344
<i>Dockstader, E. A.</i> Temperature in Reinforced Concrete Chimney Shells	March, 153
Dorchester Rapid Transit Extension. <i>Thomas F. Sullivan</i>	Jan., 49, Sept., 327
<i>Dorr, Edgar S.</i> Rubber Pavement in Boston	Jan., 54
Earthquake Hazard, Boston's	Dec., 445
<i>Eaton, A. C.</i> New England Power Company — Davis Bridge Development	Jan., 1
Fay, Frederic H. Discussion — Modern Conceptions Concerning Foundation Engineering	Dec., 428
Flat Slabs. <i>Mark Linenthal</i>	Feb., 79
Flow in Pipes, New Theories of. <i>Karl R. Kennison</i>	Oct., 357
Footings, Bond in. <i>Edwin S. Parker</i>	I., May, 246
Footings and Retaining Walls. <i>B. A. Rich</i>	Feb., 84
Foundation Engineering, Modern Conceptions Concerning. <i>Dr. Charles Terzaghi</i>	I., Dec., 397
Framingham, Mass., Recent Additions to the Sewerage System and Disposal Works. <i>F. W. Haley</i>	I., June, 253
Freeman, John R., Fund	April, 192
French Bridges	I., Sept., 329
Fresh Water Supply for Somerset Station. <i>Dana M. Wood</i>	I., Oct., 333
Gunby, Col. Frank M. Discussion — Problems of the Construction Industry	June, 298
Hale, Richard A. Discussion — Davis Bridge Development	Jan., 41
<i>Haley, F. W.</i> Recent Additions to the Sewerage System and Disposal Works of Framingham, Mass.	June, 253
<i>Hamilton, Edward P.</i> The Development of the Science of Hydraulics	Oct., 344
<i>Hazen, Allen.</i> Discussion — Modern Conceptions Concerning Foundation Engineering	Dec., 426
<i>Henderson, W. D.</i> Structural Design Features of a Hydro-Electric Development	April, 169

	PAGE
<i>Hewins, George S.</i> Discussion — Davis Bridge Development . . .	Jan., 46, 48
Highway and Rapid Transit Problems in Boston . . .	Sept., 326
Highway Construction Contracts for State . . .	March, 153
Highway Curves, Widening and Banking . . .	I., Nov., 389
Highway Essays, Prizes for . . .	Nov., 389
Highway Operation, Some Problems of. <i>G. H. Delano</i> . . .	Nov., 365
Holtwood Power Plant, Overcoming Ice Difficulties at. <i>H. W. Lowy</i> . . .	I., March, 142
Hydraulic Power Division of N. E. L. A. Meets . . .	Feb., 94
Hydraulics, The Development of the Science of. <i>Edward P. Hamilton</i> . . .	I., Oct., 344
Hydro-Electric Development, Structural Design Features of. <i>W. D. Henderson</i> . . .	I., April, 169
Ice Difficulties, Overcoming at Holtwood Power Plant. <i>H. W. Lowy</i> . . .	I., March, 142
<i>Jackson, D. C.</i> Discussion — Davis Bridge Development . . .	Jan., 42
<i>Kennison, Karl R.</i> "Specific Speed" of a Water Wheel . . .	Jan., 52
<i>Kennison, Karl R.</i> New Theories of Flow in Pipes . . .	Oct., 357
<i>Knowlton, Howard S.</i> New England Power Interests Uniting . . .	Nov., 387
"L" Shops at Everett . . .	Feb., 95
Leasing of Public Oil Lands . . .	Oct., 359
<i>Lee, E. G.</i> Discussion — Davis Bridge Development . . .	Jan., 40, 48
Library Books for Sale . . .	May, 247
<i>Linenthal, Mark.</i> Flat Slabs . . .	Feb., 79
<i>Lowy, H. W.</i> Overcoming Ice Difficulties at Holtwood Power Plant . . .	March, 142
Mapping, Aerial . . .	May, 244
<i>Marshall, R. C., Jr.</i> Problems of the Construction Industry . . .	June, 276
Massachusetts' Highways, Motor Vehicles on' . . .	Dec., 446
<i>McKenzie, Thomas.</i> Sewerage System of the Town of Westerly, R. I. . .	March, 128
Memoirs of Deceased Members:	
<i>Charles Hamilton Parker</i> . . .	Feb., 99
<i>Walter E. Parker</i> . . .	Feb., 99
<i>Clarence Henry Rooks</i> . . .	April, 198
<i>Frederic I. Winslow</i> . . .	Feb., 98
Metropolitan Planning Division . . .	Sept., 328
Metropolitan Water District Builds Steel Pipe Line . . .	Jan., 54
Metropolitan Water Supply, Report on . . .	Dec., 440
Modern Conceptions Concerning Foundation Engineering. <i>Dr. Charles Terzaghi</i> . . .	I., Dec., 397
Discussion by <i>C. M. Spofford, Lazarus White, Allen Hazen, George Paaswell</i> and <i>F. H. Fay</i> . . .	Dec., 423
Montaup Electric Company, Fresh Water Supply for Somerset Station. <i>Dana M. Wood</i> . . .	I., Oct., 333
<i>Moore, Lewis E.</i> Discussion — Problems of Highway Operation . . .	Nov., 376
Motor Vehicles on Massachusetts' Highways . . .	Dec., 446

	PAGE
Neponset Bridge, Construction of. <i>J. Stuart Crandall</i>	I., May, 232
New England Power Company — Davis Bridge Development. <i>A. C. Eaton</i>	I., Jan., 1
Discussion by <i>E. H. Rogers, H. K. Barrows, A. L. Shaw, E. G. Lee, Dwight Porter, R. A. Hale, D. C. Jackson, H. C. Whitmore, R. D. Chase and G. S. Hewins</i>	Jan., 33
New England Power Interests Uniting. <i>Howard S. Knowlton</i>	Nov., 387
New York City, The Rapid Transit System. <i>Robert Ridgway</i>	I., Sept., 305
<i>Nichols, Charles E.</i> Beams and Columns	Feb., 66
<i>Nichols, Philip.</i> Protecting the City Plan	Oct., 355
<i>Nye, George H.</i> Discussion — Some Problems of Highway Operation	Nov., 383, 384
P <i>Paaswell, George.</i> Discussion — Modern Conceptions Concerning Foundation Engineering	Dec., 428
<i>Parker, Charles Hamilton.</i> Memoir	Feb., 99
<i>Parker, Edwin S.</i> Bond in Footings	May, 246
<i>Parker, Walter E.</i> Memoir	Feb., 99
<i>Peabody, Dean, Jr.</i> The Making of Concrete	Feb., 63
Periodical Indexes, Memorandum on	June, 302
Pipe Line — Builds Ten Miles of 60 and 56 inch	Jan., 54
Planning Board, To Co-operate with	Feb., 89
Planning Boards Conference	Oct., 358
<i>Porter, Dwight.</i> Discussion — Davis Bridge Development	Jan., 41
Precise Levels at Springfield, Mass.	June, 301
Preservative Treatment of Ties on the Boston and Maine Railroad. <i>Frank C. Shepherd</i>	I., March, 101
Prizes Offered for Highway Essays	Nov., 389
Problems of Highway Operation, Some. <i>G. H. Delano</i>	Nov., 365
Discussion by <i>L. E. Moore, C. B. Breed, F. O. Whitney, G. H. Nye, R. W. Coburn and S. H. Thorndike</i>	Nov., 376
Problems of the Construction Industry. <i>R. C. Marshall, Jr.</i>	June, 276
Discussion by <i>L. C. Wason, W. F. Williams and F. M. Gunby</i>	June, 290
Proceedings of the Society: Jan., 57; Feb., 95; March, 156; Apr., 194; May, 249; June, 303; Sept., 330; Oct., 360; Nov., 393; Dec., 447.	
Publications, Three Useful	Feb., 90
Publications, Two New	May, 245
R Rainfall in Massachusetts in 1924	Feb., 94
Rapid Transit — Boston	June, 326
Rapid Transit — Dorchester	Jan., 49, Sept., 327
Rapid Transit System of New York City. <i>Robert Ridgway</i>	I., Sept., 305
Recent Additions to the Sewerage System and Disposal Works of Framingham, Mass. <i>F. W. Haley</i>	I., June, 253
Report of the Joint Committee on Standard Specifications for Con- crete and Reinforced Concrete. Discussion by <i>Dean Peabody, Jr., C. E. Nichols, Hale Sutherland, W. W. Clifford, Mark Linenthal, B. A. Rich and J. R. Worcester</i>	I., Feb., 63

	PAGE
Report on Metropolitan Water Supply	Dec., 440
Reports at Annual Meeting	April, 200
Retaining Walls, Footings and <i>B. A. Rich</i>	Feb., 84
<i>Rich, B. A.</i> Footings and Retaining Walls	Feb., 84
<i>Ridgway, Robert.</i> The Rapid Transit System of New York City	Sept., 305
<i>Rogers, Edwin H.</i> Discussion — Davis Bridge Development	Jan., 33
<i>Rogers, Edwin H.</i> Some Random Notes on Transportation	April, 161
<i>Rooks, Clarence Henry.</i> Memoir	Apr., 198
Rubber Pavement in Boston. <i>Edgar S. Dorr</i>	I., Jan., 54
Run-Off, Cycles of and Future Trends. <i>Dana M. Wood</i>	I., March, 151
Section Prizes Offered	June, 300
Sections, Are you a Member?	June, 301
Sewerage System and Disposal Works of Framingham, Mass. <i>F. W. Haley</i>	I., June, 253
Sewerage System of Westerly, R. I. <i>Thomas McKenzie</i>	I., March, 128
<i>Shaw, Arthur L.</i> Discussion — Davis Bridge Development	Jan., 39, 45
<i>Shepherd, Frank C.</i> Preservative Treatment of Ties on the Boston and Maine Railroad	March, 101
Somerset Station, Montaup Electric Company, Fresh Water Supply for. <i>Dana M. Wood</i>	I., Oct., 333
Specifications for Concrete and Reinforced Concrete, Discussions of Committee Report	I., Feb., 63
<i>Spofford, Prof. Charles M.</i> Discussion — Modern Conceptions Concerning Foundation Engineering	Dec., 423
Springfield, Mass., Precise Levels	June, 301
Stacks, Supports for Superimposed. <i>Walter W. Clifford</i>	I., May, 221
Standards, A. S. T. M.	Dec., 447
Structural Design Features of a Hydro-Electric Development. <i>W. D. Henderson</i>	I., April, 169
<i>Sullivan, Thomas F.</i> Dorchester Rapid Transit Extension	Jan., 49
Supports for Superimposed Stacks. <i>Walter W. Clifford</i>	I., May, 221
<i>Sutherland, Hale.</i> Diagonal Tension and Shear in Beams	Feb., 71
Temperatures in Reinforced Concrete Chimney Shells. <i>E. A. Dockstader</i>	I., March, 153
Temple Bill, President Coolidge Signs	March, 150
<i>Terzaghi, Dr. Charles.</i> Modern Conceptions Concerning Foundation Engineering	I., Dec., 397
<i>Thorndike, Sturgis H.</i> Discussion — Some Problems of Highway Operation	Nov., 385
Ties, Preservative Treatment on the Boston and Maine Railroad. <i>Frank C. Shepherd</i>	I., March, 101
Transportation, Some Random Notes on. <i>E. H. Rogers</i>	April, 161
Utilities Power Company of Meredith, N. H., Hydro-Electric Development. <i>William D. Henderson</i>	I., April, 169

	PAGE
<i>Wason, Leonard C.</i> Discussion — Problems of the Construction Industry	June, 290
Water Supply for Somerset Station, Montaup Electric Co. <i>Dana M. Wood</i>	I., Oct., 333
Water Supply, Report on Metropolitan	Dec., 440
Water Wheel, "Specific Speed." <i>Karl R. Kennison</i>	Jan., 52
<i>Weaver, Frederic N.</i> French Bridges	Sept., 329
Westerly, R. I., Sewerage System of. <i>Thomas McKenzie</i>	I., March, 128
<i>White, Lazarus.</i> Discussion — Modern Conceptions Concerning Foundation Engineering	Dec., 424
<i>Whitmore, Harold C.</i> Discussion — Davis Bridge Development	Jan., 43, 46
<i>Whitney, Frank O.</i> Discussion — Some Problems of Highway Operation	Nov., 381
<i>Williams, Wm. F.</i> Discussion — Problems of the Construction Industry	June, 294
Winslow, F. I. Memoir	Feb., 98
<i>Wood, Dana M.</i> Cycles of Run-Off and Future Trends	March, 151
<i>Wood, Dana M.</i> Fresh Water Supply for Somerset Station, Montaup Electric Company	Oct., 333
<i>Worcester, J. R.</i> Discussion — Report Joint Committee on Specifications for Concrete	Feb., 87

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INDEX TO ADVERTISERS

	PAGE
ADAMS, H. S., 100-1 Ames Building, Boston	iii
ASPINWALL & LINCOLN, 46 Cornhill, Boston	iii
ASSOCIATED ARCHITECTS PRINTING & SUPPLY CO., 25 Pearl St., Boston	x
BARBOUR AND DIXON, Tremont Building, Boston	iv
BARROWS, H. K., 6 Beacon St., Boston	iii
BAY STATE DREDGING AND CONTRACTING CO., 62 Condor St., East Boston	x
BEATTIE, ROY H., 209 Bedford St., Fall River	ix
BERRY DRAFTING SERVICE, 101 Tremont St., Boston	iv
BISHOP, J. W., & Co., 683 Atlantic Ave., Boston	iv
BLAIR, ISAAC, & Co., INC., 433 Harrison Ave., Boston	xii
BOSTON INSULATED WIRE & CABLE CO., Dorchester	vi
BRUNO & PETITTI, CONTRACTORS, 18 Tremont St., Boston	iv
CLIFFORD & ROEBLAD, 101 Tremont St., Boston	iv
COLEMAN BROS., 245 State St., Boston	ix
CONANT MACHINE CO., Concord Jct., Mass.	ix
CRANDALL ENGINEERING CO., 102 Border St., East Boston	xi
EASTERN BRIDGE AND STRUCTURAL CO., Worcester	viii
FAY, SPOFFORD & THORNDIKE, 200 Devonshire St., Boston	iii
FULLER & MCCLINTOCK, 170 Broadway, New York	iii
GIANT PORTLAND CEMENT CO., Philadelphia, Pa.	ix
GIBSON, RICHARD, & SON, 65 Long Wharf, Boston	ix
GOW, CHAS. R., Co., 957 Park Square Building, Boston	vi
GRANITE PAVING BLOCK MFRS.' ASSOC., 31 State St., Boston	xiv
HALL, JOHN G., & Co., 114 State St., Boston	ix
HOLZER, U., INC., 333 Washington St., Boston	vii
JACKSON & MORELAND, Park Square Building, Boston	iii
KEATING, P. J., COMPANY, 280 Main St., Fitchburg, Mass.	xii
MAIN, CHARLES T., 200 Devonshire St., Boston	ii
METCALF & EDDY, 14 Beacon St., Boston	iii
MONKS & JOHNSON, 99 Chauncy St., Boston	iv
NEW ENGLAND FOUNDATION CO., INC., 120 Tremont St., Boston	ix
OLD CORNER BOOK STORE, 50 Bromfield St., Boston	ix
PALMER CO., INC., JOHN E., 1012 Old South Building, Boston	ix
PENNSYLVANIA CEMENT CO., 161 Devonshire St., Boston	viii
PERRIN, SEAMANS & Co., 57 Oliver St., Boston	vii
PORTLAND STONE WARE CO., 49 Federal St., Boston	v
RIDEOUT, CHANDLER & JOYCE, 178 High St., Boston	ix
ROCKPORT GRANITE CO., Rockport	v
SCULLY CO., 118 First St., Cambridge	vii
SIMPSON BROS. CORPN., 77 Summer St., Boston	v
SMITH, B. F., & Co., Merchants Bank Building, 79 Milk St., Boston	ix

	PAGE
STANDARD OIL CO. OF NEW YORK, 31 St. James Ave., Boston	xii
STONE & WEBSTER, INC., 147 Milk St., Boston	xiii
STUART, T., & SON CO., Newton, Mass.	xi
THOMPSON & LICHTNER, THE, CO., INC., 80 Federal St., Boston	iii
THORPE, LEWIS D., 200 Devonshire St., Boston	iii
TUCKER, EDWARD A., CO., 101 Milk St., Boston	iv
WALDO BROTHERS AND BOND CO., 202 Southampton St., Boston	viii
WARREN FOUNDRY AND PIPE CO., 201 Devonshire St., Boston	vii
WESTON & SAMPSON, 14 Beacon St., Boston	iii
WHITMAN & HOWARD, 220 Devonshire St., Boston	iii
WORCESTER, J. R., & CO., 79 Milk St., Boston	iii
WRIGHT & POTTER PRINTING CO., 32 Derne St., Boston	x

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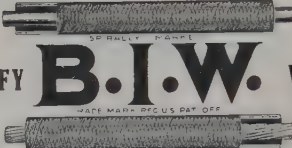
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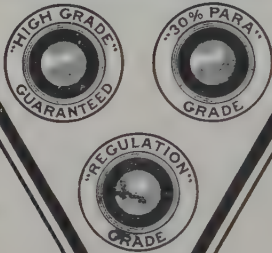
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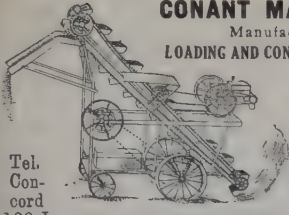
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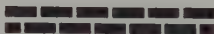
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